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PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL. XXXII. No. 1.

JANUARY, 1906.

Edited by the Secretary, under the direction of the Committee on Publications.

CONTENTS.

NEW YORK 1906.

American Society of Civil Engineers.

OFFICERS FOR 1906.

President, FREDERIC P. STEARNS.

Vice-Presidents.

Term expires January, 1907:	*Term expires January, 1908:*
M. L. HOLMAN,	ONWARD BATES,
E. KUICHLING.	BERNARD R. GREEN.

Secretary, CHARLES WARREN HUNT.

Treasurer, JOSEPH M. KNAP.

Directors.

Term expires January, 1907:	*Term expires January, 1908:*	*Term expires January, 1909.*
CHARLES S. GOWEN,	AUSTIN L. BOWMAN,	GEORGE GIBBS,
NELSON P. LEWIS,	MORRIS R. SHERRERD,	J. WALDO SMITH,
JOHN W. ELLIS,	HEZEKIAH BISSELL,	EMIL SWENSSON,
GEORGE S. WEBSTER,	EDWIN A. FISHER,	JAMES M. JOHNSON,
RALPH MODJESKI,	WILLIAM B. LANDRETH,	WYNKOOP KIERSTED,
CHARLES D. MARX.	GEORGE S. PIERSON.	WILLIAM B. STOREY, JR.

Assistant Secretary, T. J. McMINN.

Standing Committees.

THE PRESIDENT OF THE SOCIETY IS *ex-officio* MEMBER OF ALL COMMITTEES.

On Finance:	*On Publications:*	*On Library:*
EMIL KUICHLING,	MORRIS R. SHERRERD,	NELSON P. LEWIS,
CHARLES S. GOWEN,	J. WALDO SMITH,	AUSTIN L. BOWMAN,
GEORGE GIBBS,	ONWARD BATES,	RALPH MODJESKI,
M. L. HOLMAN,	BERNARD R. GREEN,	HEZEKIAH BISSELL,
GEORGE S. PIERSON.	GEORGE S. WEBSTER.	CHARLES WARREN HUNT.

Special Committees.

ON UNIFORM TESTS OF CEMENT:—George S. Webster, Richard L. Humphrey, George F. Swain, Alfred Noble, Louis C. Sabin, S. B. Newberry, Clifford Richardson, W. B. W. Howe, F. H. Lewis.

ON RAIL SECTIONS:—Joseph T. Richards, C. W. Buchholz, E. C. Carter, S. M. Felton, Robert W. Hunt, John D. Isaacs, Richard Montfort, H. G. Prout, Percival Roberts, Jr., George E. Thackray, Edmund K. Turner, William R. Webster.

ON CONCRETE AND REINFORCED CONCRETE:—C. C. Schneider. J. E. Greiner, W. K. Hatt, Olaf Hoff, Richard L. Humphrey, Robert W. Lesley, J. W. Schaub, Emil Swensson, A. N. Talbot. J. R. Worcester.

The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER: - - - 533 Columbus.
CABLE ADDRESS: - - - "Ceas, New York."

Vol. XXXII. JANUARY, 1906. No. 1.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS.

CONTENTS:

MINUTES OF MEETINGS.

OF THE SOCIETY.

January 3d, 1906.—The meeting was called to order at 8.35 P. M.,; President C. C. Schneider in the chair; Chas. Warren Hunt, Secretary; and present, also, 84 members and 22 guests.

The minutes of the meetings of December 6th and 20th, 1905, were approved as printed in the *Proceedings* for December, 1905.

A paper by Albert J. Himes, M. Am. Soc. C. E., entitled "The Position of the Constructing Engineer, and his Duties in Relation to Inspection and the Enforcement of Contracts," was presented by the author. Written communications from Messrs. James Smith Haring, W. D. Lovell, Benjamin Thompson, S. Bent Russell and

Willard Beahan were presented by the Secretary, and the subject was discussed verbally by Messrs. W. A. Aiken, G. S. Bixby, Augustus Smith and the author.

Ballots for membership were canvassed, and the following candidates elected:

As Members.

Thomas David Allin, Pasadena, Cal.
James Leland Crider, Mt. Vernon, N. Y.
William Herbert Cushman, Huntingdon, Pa.
Thomas Gregory Dabney, Clarksdale, Miss.
Edward John Duffies, Harbor Beach, Mich.
Chester Shepard Freeland, San Francisco, Cal.
Edward Payson Lupfer, Buffalo, N. Y.
William Vaughan Polleys, Providence, R. I.
Victor da Silva Freire, Sao Paulo, Brazil.
Herman Winslow Spooner, Gloucester, Mass.

As Associate Members.

Edward Maguire Adams, Fort Leavenworth, Kans.
Thomas K Bell, Philadelphia, Pa.
Paul Alexander Blackwell, Canonsburg, Pa.
Harry Westbrook DeGraff, Fonda, N. Y.
Osborne Joel Dempster, Albany, N. Y.
Lorenzo Carlisle Dilks, New York City.
Elmer Dwight Harshbarger, Aspinwall, Pa.
Frank Edward Hermanns, New York City.
Walter Burditt Leane, Cape Colony, South Africa.
Alfred Emanuel Lindau, St. Louis, Mo.
Alexander Majors Munn, Nebraska City, Nebr.
John Peterson, New York City.
Arthur Monroe Shaw, Dixon, Ill.
Edward Clayton Sherman, Boston, Mass.
Charles Bailey Smith, Boise, Idaho.
Raymond French Stoddard, Milford, Conn.
Samuel Forsythe Thomson, New York City.

As Associates.

John Moffatt Bruce, New York City.
William Roberts Conard, Burlington, N. J.

The Secretary announced the transfer of the following candidates, by the Board of Direction, on January 2d, 1906:

From Associate Member to Member.

William George Brenneke, St. Louis, Mo.
Edward Bayrd Fay, St. Louis, Mo.

The election of the following candidates, by the Board of Direction, on January 2d, 1906:

As Juniors.

Duff Andrew Abrams, Champaign, Ill.
Arthur Boniface, New York City.
Arthur Robert Brown, Bas Obispo, Canal Zone, Panama.
Howard Foss Esten, Pawtucket, R. I.
Walter Gottlieb Federlein, New York City.
James Stanley Frazer, Mt. Vernon, N. Y.
Ray Palmer Hovey, Providence, R. I.
Charles Hamilton Lee, Los Angeles, Cal.
Henry Schell Nichol, New York City.
Alfred Moore O'Neal, Jr., New York City.
David Elliott Pendleton, Greenville, Miss.
William Jenner Powell, Empire, Canal Zone, Panama.
Charles Hinkley Van Kirk, Chicago, Ill.
Alfred Marshall Wyman, East Orange, N. J.

The Secretary announced the following deaths:

Anthony Houghtaling Blaisdell, elected Member, March 3d, 1880; died September 9th, 1905.

James MacNaughton, elected Member, May 5th, 1880; died December 29th, 1905.

Adjourned.

January 17th, 1906.—The minutes of this, the Annual Meeting, will be printed in the *Proceedings* for February, 1906.

ANNOUNCEMENTS.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

MEETINGS.

Wednesday, February 7th, 1906.—8.30 P. M.—A regular business meeting will be held. Ballots for membership will be canvassed, and a paper, entitled "Test of a Three-Stage, Direct-Connected Centrifugal Pumping Unit," by Philip E. Harroun, M. Am. Soc. C. E., will be presented for discussion.

This paper was printed in *Proceedings* for December, 1905.

Wednesday, February 21st, 1906.—8.30 P. M.—At this meeting a paper, entitled "The Economical Design of Reinforced Concrete Floor Systems for Fire-Resisting Structures," by John S. Sewell, M. Am. Soc. C. E., will be presented for discussion.

This paper was printed in *Proceedings* for December, 1905.

Wednesday, March 7th, 1906.—8.30 P. M.—A regular business meeting will be held. Ballots for membership will be canvassed, and a paper, entitled "The Theory of Continuous Columns," by Ernst F. Jonson, Assoc. M. Am. Soc. C. E., will be presented for discussion.

This paper is published in this number of *Proceedings.*

Wednesday March 21st, 1906.—8.30 P. M.—At this meeting a paper, entitled "New Facts about Eye-Bars," by Theodore Cooper, M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings.*

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all meetings:

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Society of Engineers, 17 Victoria Street, Westminster, S. W., England.

American Institute of Mining Engineers, 99 John Street, New York City.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Civil Engineers' Club of Cleveland, 1200 Scofield Building, Cleveland, Ohio.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Philadelphia, 1122 Girard Street, Philadelphia, Pa.

Engineers' Society of Western Pennsylvania, 410 Penn Avenue, Pittsburg, Pa.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Engineers' Club of Central Pennsylvania, Corner, Second and Walnut Streets, Harrisburg, Pa.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Teknisk Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Société des Ingénieurs Civils de France, 19 Rue Blanche, Paris, France.

Svenska Teknologförenigen, Brunkebergstorg 18, Stockholm, Sweden.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Sachsischer Ingenieur= und Architekten= Verein, Dresden, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Pacific Northwest Society of Engineers, 617–618 Pioneer Building, Seattle, Wash.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Memphis Engineering Society, Memphis, Tenn.

Oesterreichischer Ingenieur= und Architekten=Verein, Eschenbachgasse 9, Vienna, Austria.

The Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Cleveland Institute of Engineers, Middlesborough, England.

SEARCHES IN THE LIBRARY.

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many such searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling, compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

Copies of all lists of references are filed, so that in many cases it is only necessary to make a typewritten copy, which reduces the cost of searches to a minimum.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

ANNUAL REPORT OF THE BOARD OF DIRECTION FOR THE YEAR ENDING DECEMBER 31st, 1905.

PRESENTED AT THE ANNUAL MEETING, JANUARY 17TH, 1906.

The Board of Direction, in compliance with the Constitution of the Society, presents its report for the year ending December 31st, 1905.

MEMBERSHIP.

The changes in membership are shown in the following table:

Grade.	Jan. 1st, 1905.			Jan. 1st, 1906.			Losses.				Additions.		Totals.	
	Resident.	Non-Resident.	Total.	Resident.	Non-Resident.	Total.	Transfer.	Resignation.	Dropped.	Death.	Transfer.	Election.	Loss.	Gain.
Honorary Members	1	8	9	1	11	12	...	..	..	..	2	1	....	3
Corresponding Members	....	2	2	....	2	2	...	..	..	..	....	....	....	....
Members	376	1 419	1 795	420	1 532	1 952	2	3	1	24	*68	119	30	187
Associate Members	240	663	903	285	736	1 021	64	3	4	8	†44	‡153	79	197
Associates	51	76	127	52	79	131	2	4	1	1	....	12	8	12
Juniors	116	224	340	119	277	396	46	1	2	3	....	108	52	108
Fellows	9	18	27	9	16	25	...	..	..	2	....	....	2	....
Total	793	2 410	3 203	886	2 653	3 539	114	11	8	38	114	393	171	507

* 64 Associate Members. 1 Associate and 3 Juniors.
† 1 Associate and 43 Juniors.
‡ 2 Reinstatements.

It will be seen that the net increase during the year has been 336, which is 57 greater than ever before. The yearly net gains in total membership, beginning with 1899, are given below. Prior to that year the yearly increase reached 100 only four times, no two of which were consecutive.

Year.	Net increase.
1899	103
1900	138
1901	190
1902	185
1903	212
1904	279
1905	336
Total net increase in seven years	1 443
Average net yearly increase	206

For comparison, the average net increase previous to 1899 is given below.

Average net yearly increase	1870-78	48.1
" " " "	1879-88	61.1
" " " "	1889-98	89.3

The total number of applications received during the year was 549.

The losses by death reported during the year number 38. They are as follows:

Members: Julius Baier, Burr Bassell, Anthony Houghtaling Blaisdell, Robert Cartwright, George William Catt, Casimir Constable, Frederick de Funiak, George William Frank, Edward Sherman Gould, David Maxson Greene, Frank March Haines, Richard Somers Hayes, George Anthony Lederle, Gabriel Leverich, Thomas John Long, James MacNaughton, George Anson Marr, Edmund Trowbridge Dana Myers, William Beswick Myers-Beswick, John Talcott Norton, Sutherland Mallet Prevost, William Marshall Rees, Archer Cochran Stites, Nathan Hollis Whitten.

Associate Members: Justin Burns, Frank Lewis Fales, Van Dusen Hite-Smith, Louis Ralph Lavalle, Henry Brigham Looker, William Smith Morison, Macy Stanton Pope, George Draper Stratton.

Associate: Cassius Howard Lindenberger.

Juniors: George Wallace Enos, Shukichi Fujino, George Clifton Woollard.

Fellows: Thomas Cooper Coleman, George Clarke Walker.

LIBRARY.

The accessions to the Library during the year are shown in the following table:

ACCESSIONS DURING THE YEAR 1905.

	Bound Volumes.	Unbound Volumes.	Specifications.	Maps, Photographs and Drawings.	Total.
Donations—					
In answer to special requests	289	463	...	147	899
From publishers	61	9	...	...	70
In regular course	504	1 292	267	116	2 179
By purchase	142	27	...	...	169
Totals	996	1 791	267	263	3 317

In addition to the above, there have been received 781 duplicates, and 15 separate numbers to complete files of periodicals, neither of which can appear as accessions.

The total number of titles in the Library is 21 083.

The Library now contains:

Bound volumes	14 458
Unbound volumes	28 912
Specifications	6 269
Maps, photographs and drawings	3 730
Total	53 369

During the year, 359 volumes have been bound, and 19 bound volumes which are duplicates were received and have replaced previously unbound volumes.

The following amounts have been expended upon the Library during the year:

Purchase of books, 196 volumes	$549.84
Express charges, etc	11.72
Binding 359 volumes	424.05
Fixtures, supplies and sundries	94.37
Total	$1 079.98

The value of the accessions to the Library during the year is as follows, each accession having been valued separately, as received:

3 149 Donations and exchanges (estimated value)	$2 295.53
169 Volumes purchased (cost)	442.52
Binding 359 volumes	424.05
Total	$3 162.10

During the year 2 869 persons have used the Reading Room and Library.

Searches have been made for 54 persons, covering 825 separate references. These searches, the expense of which is trifling, seem to be of great convenience and value to those who ask for them.

During the year, as heretofore, every effort has been made to secure by gift, if possible, and, if not, by purchase, every engineering work published which should be on our shelves.

From time to time members have suggested the advisability of making the Library a circulating one, and this has been frequently considered by the Board. The difficulties in the way of such a project, however, including the prohibitive cost of securing at least

one or more duplicates of all books of reference, seem to be insurmountable.

PUBLICATIONS.

During the year the usual ten numbers of *Proceedings* and two volumes of *Transactions* have appeared.

In the *Proceedings*, the list of references to current engineering literature has covered 113 pages, containing 5 430 classified references to 83 periodicals. Last year only 71 periodicals were reviewed.

The stock of the various publications of the Society, kept on hand for the convenience of members and others, now amounts to 136 416 copies, the cost of which to the Society, for paper and presswork only, has been $18 574.86.

During the year, 11 402 volumes of the Society publications have been bound, for members and others, in the standard half-morocco or cloth bindings.

The extra publications during the year have been six volumes of *Transactions* containing the papers and discussions of the International Engineering Congress. Details as to this publication are given under another heading.

Summary of Regular Publications for 1905.

	Issued.	Edition.	Total Pages.	Plates.	Cuts.
Transactions (Volumes)*....	2	3 725	1 034	127	160
Proceedings (Monthly Numbers)..................	10	3 375	1 271	104	107
Constitution and List of Members..................	1	4 200	248	...	1
Total.................	13	....	2 553	231	268

The cost of regular publications has been:

For Paper, Printing, Binding, etc., *Transactions* and *Proceedings*	$11 375.45
For Plates and Cuts................................	1 733.78
For Boxes, Mailing Lists, Copyright and Sundry Expenses	375.97
For 5 940 copies of Memoirs and Papers..............	741.03
For List of Members................................	1 147.90
Total..................................	$15 374.13
Deduct amount received from sale of publications......	2 501.98
Net cost of Regular Publications for 1905........	$12 872.15

* Includes Indexes and Tables of Contents.

PUBLICATIONS OF THE INTERNATIONAL ENGINEERING CONGRESS.

The last report of the Board contained an outline of the work of the International Engineering Congress from its inception in September, 1903, to January 1st, 1905. At that date the Board reported that the work of preparing the discussions was in progress, and that it was hoped to have the whole Congress Publication ready for issue before July 1st, 1905. This expectation was realized. The discussions were collated, put in type, forwarded to the Author of each paper, and, with his closure, published with the paper. All papers and discussions were grouped by subjects. The result is embraced in six volumes, issued, for convenience, as Parts A, B, C, D, E and F of Volume LIV of *Transactions*, and your Board believes that the quality and extent of this publication fully justified the labor and expense which was assumed by the Society.

These extra volumes were furnished without cost to all members, and were also furnished to 335 subscribers, 192 of whom paid $5, and 143, $10. A number of sets have also been sold at $5 per volume. Persons who joined the Society after January 1st, 1905, may secure these volumes at a special price of $2.50 per volume.

Summary of Publications of the International Engineering Congress, 1905, *Transactions*, Volume LIV, Parts A, B, C, D, E and F.

Number of Volumes	6	
Edition	4 000	
Total Pages	3 398	
Number of Plates	248	
" " Cuts	441	
" " Subjects treated	36	
" " Papers	96	
" " Discussions	302	
Separate Papers Printed	19 675	
The total cost of the Congress has been		$38 462.80
Amount expended in 1904 for this account		13 579.27
Amount expended during 1905		$24 883.53
There has been received from subscriptions and sales of publications of the Congress		$4 949.05

Owing to these extra publications, the work of the staff of the Society was materially increased during the year. An idea may be

conveyed by the statement that of the publications alone more than 80 000 separate pieces were handled.

LOCAL ASSOCIATIONS OF MEMBERS.

The formation of Local Associations of Members of the Society in the larger cities of the country has received much attention during the past year. The Board prepared a report on the subject which was presented to the Society at the Annual Convention at Cleveland, where the subject was fully discussed.

Local Associations of Members of the Society have been successfully organized at Kansas City, Mo., San Francisco, Cal., and Memphis, Tenn., and much interest has been manifested in other centers.

MEETINGS.

During the year, 24 meetings have been held, as follows: Annual Meeting, 2; at the Annual Convention, 4; regular semi-monthly meetings, 18.

At these meetings there were presented 21 formal papers, two of which were illustrated with lantern slides, and two illustrated lectures.

The Thirty-seventh Annual Convention was held at Cleveland, Ohio, at which the registered attendance numbered 320 members and 255 guests.

MEDALS AND PRIZES.

For the year ending with the month of July, 1904, Prizes were awarded as follows:

The Norman Medal to Emile Low, M. Am. Soc. C. E., for his paper entitled "The Breakwater at Buffalo, N. Y."

The Thomas Fitch Rowland Prize to George Cecil Kenyon, Assoc. M. Am. Soc. C. E. (now M. Am. Soc. C. E.), for his paper entitled "Dock Improvements at Liverpool."

The Collingwood Prize for Juniors to Herbert J. Wild, Jun. Am. Soc. C. E. (now Assoc. M. Am. Soc. C. E.), for his paper entitled "The Substructure of Marsh River Bridge."

SOCIETY HOUSE.

In its last report the Board informed the membership of the purchase of the 25-ft. lot adjoining the Society House, and stated that general plans were being prepared with the expectation that the work of enlargement would go on during 1905.

The Board now reports that plans for the enlargement were prepared by Messrs. Eidlitz and McKenzie early in the year. Competitive bids were secured, and on May 2d, 1905, a contract was signed with William L. Crow amounting to $52 497.

Owing to the difficulty of getting steel and to some minor labor troubles, the work is not completed, but it is expected at this writing that the building will be far enough advanced to enable the Annual Meeting to be held in it.

In a general way, the addition provides an enlargement of 50% in every department of the Society's work. The Lounging Room and Auditorium are each increased in length 25 ft. An additional Reading Room is provided on the second floor, and the capacity of the offices on the third floor, as well as the Stack Room on the fourth floor, has been materially increased.

The rooms which were used in the old building as offices of the Secretary have been added to the entrance hallway, forming a new Reception Hall, and the entrance to the Lounging Room as well as the toilet facilities have been enlarged.

A stairway leading from the Lounging Room to the Auditorium has been provided, which it is believed will add much to the facilities of the building, and to the comfort of those in attendance at meetings.

The building operations have been in the hands of a Committee consisting of Messrs. Noble, Deyo, N. P. Lewis and Hunt. As soon as a final settlement is made, the report of this Committee will be published in *Proceedings* for the information of members.

FINANCES.

The attention of Members is invited to the Secretary's statement of receipts and disbursements, and to the general balance sheet which accompanies it, in which the very satisfactory financial condition of the Society appears.

In a circular issued by the Board of Direction dated May 25th, 1895, when the building of a Society House was first contemplated, the available assets of the Society were stated to be as follows:

"The House 127 East 23d St. (estimate)....	$60 000
"Securities in Safe Deposit (par value)....	16 000
"Cash awaiting permanent investment........	4 500
"Total....................................	$80 500
"Mortgage on 127 E. 23d St......	16 000
"Amount Available..............	$64 500"

When the enlargement of the Society House is finished and paid for, a similar statement will be about as follows:

Lot (75 by 112) 218-222 W. 57th St. (estimate)	$270 000.00
House (1897) Cost	103 597.83
Addition (1905-06) (estimate)	60 000.00
Total	$433 597.83
Mortgage and Loan	190 000.00
Total value of real property, above debt.	$243 597.83

It should be noted that the above statement relates only to the real property of the Society, and does not show the increased value of the Library nor the amount expended for furniture, etc.

In closing this report the Board desires to call attention to the generous donation of Thomas Fitch Rowland, Hon. M. Am. Soc. C. E. Resolutions of thanks have already been adopted by the Board and by the Society, but the Board wishes to emphasize in this report its high appreciation of this timely contribution to the funds of the Society.

The reports of the Secretary and Treasurer are appended.

By order of the Board of Direction.

CHAS. WARREN HUNT,
Secretary.

NEW YORK, JANUARY 2D, 1906.

REPORT OF THE TREASURER.

In compliance with the provisions of the Constitution, the Treasurer presents the following report for the year ending December 31st, 1905:

Balance on hand December 31st, 1904			$49 927.49
Receipts from current sources, January 1st to December 31st, 1905			81 742.14
Extraordinary Receipts on account of:			
Donation	$5 000.00		
International Engineering Congress	2 333.90		
St. Louis Exhibit and Headquarters	195.89		
Entertainment of British Engineers	80.00		
Additional Loan	3 000.00		
			10 609.79
Payment of Audited Vouchers for Current Business, January 1st to December 31st, 1905		$67 296.94	
Extraordinary Expenses on account of:			
Building	$36 919.80		
International Engineering Congress	24 932.93		
Entertainment of British Engineers	172.50		
		62 025.23	
Balance on hand December 31st, 1905:			
In Union Trust Company.	$5 395.51		
In Garfield National Bank.	6 061.74		
In hands of the Treasurer.	1 500.00	12 975.25	
		$142 279.42	$142 279.42

Respectfully submitted,

JOS. M. KNAP,

Treasurer, Am. Soc. C. E.

New York, January 2d, 1906

REPORT OF THE SECRETARY, FOR THE

To the Board of Direction of the

Gentlemen:—I have the honor to present a statement of Re-
December 31st, 1905. I also append a general balance sheet show-

New York, January 2d, 1905.

Receipts.

Balance on hand December 31st, 1904, in Bank, Trust Company and in hands of Treasurer............		$49 927.49
Entrance Fees........................	$9 665.00	
Current Dues........................	37 642.58	
Past Dues...........................	1 519.71	
Advance Dues........................	17 595.11	
Certificates of Membership............	352.75	
Badges	2 014.50	
Sales of Publications..................	2 501.98	
Interest	987.07	
Library	272.85	
Convention	507.00	
Annual Meeting.......................	894.00	
Binding	7 676.80	
Miscellaneous	112.79	
Donation	5 000.00	
International Engineering Congress.....	2 333.90	
Louisiana Purchase Exposition.........	195.89	
Entertainment of British Engineers.....	80.00	
Loan and Mortgage....................	3 000.00	
		$92 351.93
		$142 279.42

YEAR ENDING DECEMBER 31st, 1905.

American Society of Civil Engineers.

ceipts and Disbursements for the fiscal year of the Society, ending ing the condition of the affairs of the Society.

Respectfully submitted,

CHAS. WARREN HUNT,
Secretary.

Disbursements.

Salaries of Officers	$8 500.00	
Clerical Help	12 323.35	
Caretaking	1 771.94	
Publications	15 374.13	
Postage	3 763.33	
General Printing and Stationery	2 324.51	
Badges	1 636.90	
Certificates of Membership	222.70	
Binding	6 542.70	
Library	1 079.98	
Maintenance of House	285.80	
Heat, Light and Water	1 116.50	
Furniture	201.50	
Annual Meeting	1 528.25	
Convention	1 131.51	
Prizes	179.85	
Interest and Insurance	8 676.98	
Petty Expenses	185.49	
Entertainment of British Engineers	172.50	
Building	36 888.55	
Louisiana Purchase Exposition	31.25	
Refunds	10.35	
Current Business	441.17	
International Engineering Congress	24 932.93	
		$129 322.17
Balance on hand December 31st, 1905:		
In Union Trust Company	$5 395.51	
In Garfield National Bank	6 061.74	
In hands of Treasurer	1 500.00	
		$12 957.25
		$142 279.42

ACCOMPANYING THE REPORT OF THE SECRETARY.

Assets.		
Three Lots (estimated value)		$270 000.00
Society Building (cost)		140 486.38
Furniture (cost)		16 534.94
Publications on hand (inventoried cost, value)		18 574.86
Library:		
Cash expended for Books, etc.	$10 400.03	
Donations, estimated value	47 055.59	
		57 455.62
Due from Members	$4 265.29	
" " Non-Members	415.43	
		4 680.72
Cash		12 957.25
		$520.689.77

Liabilities.		
Dues for 1906, paid in advance		$17 595.11
Mortgage Debt and Loan		178 000.00
*Funds invested in Society House, Lots and Library.	$26 730.78	
Surplus	298 363.88	
		325 094.66
		$520.689.77

*These Funds have been derived from the following sources:

Norman Medal Fund	$1 000.00
Rowland Prize Fund	1 222.50
Collingwood Prize Fund	1 000.00
McCrea Swift Fund	1 000.00
Herbert Stewart Library Fund	1 940.00
Fellowship Fund	13 038.28
Compounding Dues Fund	7 530.00
	$26 730.78

The Norman Medal, Rowland Prize and Collingwood Prize Funds no longer exist as such, the obligations under them having been assumed by the Society, with the consent of the donors.

NEW YORK, 12TH JANUARY, 1906.—We have examined the books and accounts of the American Society of Civil Engineers, and certify that the foregoing Balance is in accordance therewith, as of 31st December, 1905; and, in our opinion, correctly states the condition of the Society's affairs as shewn by the books at that date.

MENZIES, FAWCETT, TOD & CO.,
Chartered Accountants.

ACCESSIONS TO THE LIBRARY.

From December 11th, 1905, to January 6th, 1906.

DONATIONS.*

MODERN TURBINE PRACTICE AND WATER POWER PLANTS.

By John Wolf Thurso. Cloth, 9 x 6 in., illus., 22 + 244 pp. New York, D. Van Nostrand Company, 1905. $2 net.

The preface states that the object of this book is to give such information in regard to modern turbines and their proper installation as is necessary to the hydraulic engineer in designing a water-power plant, and no attempt has been made to treat the design of turbines. In the first part the writer has intended to show the deficiencies of the present American turbine practice and to point out the direction in which improvement is to be sought. On account of the growing importance of the steam turbine and its close relation to the hydraulic turbine, a chapter has been included on this subject. In the second part will be found information and data relative to turbine plants. Following the preface are a few pages of terms and symbols used in hydraulic-power engineering. Contents: Part I.—Modern Turbine Practice: Turbine Practice in Europe; Turbine Practice in America; Classification of Turbines; Steam Turbines; Modern Turbine Types and Their Construction; Accessories to Turbines; Governors and Speed Regulators. Part II.—Water Power Plants: Water-Conductors; British and Metric Measures and Values. There is an index of six pages. At the end of the book is Mr. Allan V. Garratt's paper on speed regulation of turbines, reprinted from the *Transactions* of the American Institute of Electrical Engineers.

MACHINE TOOLS.

For Planing, Shaping, Slotting, Drilling, Boring, Milling, Wheel Cutting, Their Design and Construction. By Thomas Shaw. Cloth, 9 x 6 in., illus., 7 + 676 + 8 pp. Manchester, England, The Scientific Publishing Company. 15 shillings.

In this work the author aims to review the construction of the different types of machine tools in which, generally speaking, the work is either held stationary or moves in straight lines, as distinguished from those to which a direct rotary motion is given. He has endeavored to present, from actual practice, a number of examples of each type of machine, showing the different mechanisms with their respective constructional surroundings, and, in some cases, the complete arrangement of details. Besides the chapters dealing with the construction of the machines, there has been added a chapter of useful items and miscellaneous information collected from the author's personal experience and other sources. The contents are: Planing Machines; Shaping and Slotting Machines; Vertical Drilling Machines; Radial Drilling Machines; Horizontal Boring and Drilling Machines; Multiple-Spindle Drilling Machines; Milling Machines; Wheel-Cutting Machines; Items of Interest, and Appendix. There is an index of seven pages.

CONTRIBUTION À L'ÉTUDE DE LA FRAGILITÉ DANS LES FERS ET LES ACIERS.

Mémoires Originaux et Réimpressions; Publication faite avec le Concours des Six Grandes Compagnies de Chemins de Fer Français. Société D'Encouragement pour l'Industrie Nationale. Paper, 10 x 9, illus., 16 + 482 pp. Paris, Siège de la Société, 1904. $6.

This work comprises original papers and reprints on the brittleness of steel, with an introduction by H. Le Chatelier. Some of the contents are: Résistance au Choc et Fragilité du Fer, et Mesure de la Fragilité de l'Acier et du Fer, par M. Considère; Influence de la Température sur les Propriétés Mécaniques des Métaux, par M. André Le Chatelier; Fragilité après Ecrouissage à Froid et Fissilité, par M. Considère; Fragilité des Aciers, par M. Godron; Aciers Propres à la Construction des Machines, par M. Auscher; Nouvelle Méthode d'Essai des Métaux, par M. Ch. Frémont; Influence du Temps et de la Température sur les Propriétés Mêcaniques et les Essais des Mêtaux, par M. André Le Chatelier; Etude Experimental des Causes de la Fragilité de l'Acier,

*Unless otherwise specified, books in this list have been donated by the publishers.

par M. Ch. Frémont; Sur le Pliage des Barrettes Entaillées, par MM. Ch. Frémont et F. Osmond; Flexion par Choc sur Barreaux Entaillés, par M. J. Barba, et par M. G. Charpy; Essai des Métaux par Pliage de Barrettes Entaillées, par M. Ch. Frémont; Essai de Fragilité au Choc sur Barreaux Entaillés, par M. H. Le Chatelier; Note sur le Rôle des Essais dans le Controle du Material Roulant de Chemin de Fer, et L'Essai au Choc des Métaux dans la Construction du Matérial Roulant de Chemin de Fer, par M. F. Vanderheym. The book has a table of contents, but contains no index.

INDEX TO ENGINEERING NEWS FOR THE YEARS 1900 TO 1904 INCLUSIVE.

Compiled by Mary E. Miller. Cloth, 9 x 6 in., 290 pp. New York, Engineering News Publishing Company, 1905. $2.

The introduction states that the present publication, indexing the ten volumes of *Engineering News* issued from January 1st, 1900, to January 1st, 1905, is the third such general index to this journal which has been issued. The first covered the sixteen years from the foundation of the journal in 1874 to the end of 1890. The second, a much larger and more systematically compiled volume, covered the years from 1890 to 1899, inclusive; and the present, as just stated, covers the matter published in the five years following. As a slight indication of the growth of the engineering profession and the corresponding growth of *Engineering News*, it may be here noted that the first general index, covering the sixteen years preceding 1890, was a volume of 118 pages; the second book, covering the decade preceding 1900, contained 324 pages, and the present volume, covering only half a decade, contains 291 pages. In part, however, the increase in size is due to more careful and complete indexing. The present volume is designed to be a labor-saving aid in the use of *Engineering News*. The introduction also contains suggestions on how to use the index. The author index at the end of this book is a new feature, not contained in the other volumes.

A POCKET BOOK OF MECHANICAL ENGINEERING;

Tables, Data, Formulas, Theory and Examples for Engineers and Students. By Charles M. Sames. Leather, 6 x 4 in., 8 + 168 pp. Jersey City, N. J., Charles M. Sames, 1905. $1.50.

This book is the result of the writer's endeavor to compact the greater part of the reference information, usually required by mechanical engineers and students, into a volume, the dimensions of which permit of its being carried in the pocket without inconvenience. The author states that in its preparation he has consulted standard treatises and reference books, the transactions of engineering societies and his own memoranda, which extend over a period of fifteen years. A large amount of valuable and timely matter has been obtained from the columns of technical periodicals and also from the catalogues which manufacturers have courteously placed at his disposition. The contents are: Mathematics; Chemical Data; Materials; The Strength of Materials, Structures and Machine Parts; Energy and the Transmission of Power; Heat and the Steam Engine; Hydraulics and Hydraulic Machinery; Shop Data; Electrotechnics; Addenda. There is an index of six pages.

EARTHWORK TABLES.

By R. S. Henderson. Paper, 12 x 9 in., 32 pp. New York, The Engineering News Publishing Company, 1905. $1.

Part I gives Cubic Yards per 100 Feet for Level Sections; Bases up to 50 Feet varying by 1 Foot; Heights up to 25 Feet varying by 0.1 Foot; Heights, 25 Feet to 50 Feet, varying by 0.5 Foot; Heights, 50 Feet to 100 Feet, varying by 1 Foot; Quantities given direct for Slope of 1½ to 1; Quantities for 10 other Slopes given by one Addition or Subtraction; Results to the nearest Yard; To which is added a Graphical Method of Estimating Quantities from a Profile. Part II gives the Volume, in Cubic Yards, of Prismoids, 100 Feet Long, by the Average End-Area Method; sums of End Areas up to 10 000 Square Feet, varying by tenths; Cubic Yards per 100 Feet to the nearest tenth; Sums of End Areas up to 1 000 Square Feet, varying by hundredths; Cubic Yards per 100 Feet, to the nearest hundredth.

HYDROGRAPHIC SURVEYING;

Methods, Tables and Forms of Notes. By Samuel H. Lea, M. Am. Soc. C. E. Cloth, 9 x 6 in., illus., 172 pp. New York, The

Engineering News Publishing Company, 1905. $2 net (donated by the author).

The author states in the preface that this work is intended to be useful to engineering students and the younger members of the profession, as well as to those engineers of larger experience, who are not thoroughly familiar with the minor details of Hydrographic Surveying, much of the matter contained in the manual being elementary and the hydraulic formulas being stated as nearly as practicable in their simplest forms. It is the author's purpose to explain such practical features of hydrographic surveying as are likely to be encountered in actual practice and are at present to be learned only by actual field experience. It is stated that much of the information was acquired by the author in the course of his professional work, consequently, it has the merit of practicality. He has endeavored to limit the scope to such ordinary methods of hydrographic surveying as are likely to occur, and desires to embody within this little book a concise explanation of modern methods. There is no index.

MODERN MACHINE SHOP;

Construction, Equipment and Management. By Oscar E. Perrigo. Cloth, 10 x 7 in., illus., 343 pp. New York, The Norman W. Henley Publishing Company, 1906. $5.

The aim of the author has been to produce a work suitable for the practical and every-day use of the architects who design, the manufacturers who build, the engineers who plan and equip, the superintendents who organize and direct, and for every stockholder, officer and workman of the modern machine shop and manufacturing plant. Part First is devoted to the *Construction*, describing and illustrating buildings of approved form and arrangement; Part Second to *Equipment* with modern tools, machines and appliances; and Part Third discusses the question of *Management*, minutely describing and illustrating, it is stated, a plain, concise, accurate and common-sense system of management and of time and cost keeping, that may be easily and economically administered and give information necessary for operating the business to financial success. There is an index of nearly sixteen pages.

PRACTICAL PATTERN-MAKING.

By F. W. Barrows. Cloth, 7 x 5 in., illus., 326 pp. New York, The Norman W. Henley Publishing Company, 1906. $2.

It is stated that the aim of this book is to be a thoroughly practical work. It is written by a pattern-maker with thirty years' experience, and contains information on pattern-making and pattern-makers in general, and a detailed description of the necessary materials, hand and machine tools, with special chapters on the lathe, the band-saw and the circular saw, with examples of work which may be done on these machines. A complete section of illustrated examples of pattern-work in wood, with many pages of metal pattern-work, gating and plate work, both vibrator and stripping plates, are shown. Some mathematics for the pattern shop are given, and finally the cost, marking and record of patterns is explained and illustrated. There is an index of fourteen pages.

Gifts have also been received from the following:

Allen, W. F. 1 vol.
Am. Soc. for Testing Materials. 1 vol.
Am. Soc. of Heating and Ventilating Engrs. 1 vol.
Am. Water Works Assoc. 1 vol.
Atlantic Coast Line R. R. Co. 2 pam.
Bixby, G. S. 2 pam.
Brit. Fire Prevention Committee. 1 pam.
Cal. Academy of Sci. 2 pam., 1 vol.
Chicago, Ill.—Dept. of Public Works. 1 pam.
Chicago Junction Rys. & Union Stock Yards Co. 1 pam.
Colo.—State Agri. Exper. Station. 3 pam.
Cornell Univ. Library. 1 pam.
Endicott, M. T. 1 vol.
Eng. Standards Committee. 1 bound vol.
Florida—Railroad Comm. 3 pam.
Foster-Munger Co. 1 vol.
Great Britain—Patent Office. 3 vol.
Hermany, Charles. 4 vol.
Humphreys, A. C. 1 pam.
Indian Midland Ry. Co., Ltd. 1 pam.
Inst. of Engrs. and Shipbuilders in Scotland. 1 bound vol.
Kenyon, G. C. 1 pam.
Lehigh & Hudson River Ry. Co. 1 pam.
London, Ont.—Board of Water Commrs. 1 pam.
Madras, India—Public Works Dept. 5 vol.
Mahl, William. 2 vol.
Mass. Inst. of Tech. 1 vol.
National Board of Fire Underwriters Committee of Twenty. 1 vol., 2 pam.

National Irrig. Congress. 1 vol.
New South Wales—Govt. Statistician. 2 pam.
New York City—Dept. of Health. 1 pam.
New York—State Library. 1 pam.
New York—State Museum. 1 vol.
Nichols, T. F. 4 vol.
Ockerson, J. A. 1 pam.
Ohio—State Geologist. 1 pam.
Pere Marquette R. R. Co., 4 pam.
Platt, T. C. 7 vols.
Southern Pacific Co. 1 vol.
Tonindustrie-Zeitung. 1 bound vol., 2 vol.
U. S. Bureau of the Census. 1 bound vol.
U. S. Corps of Engrs. 23 specif.
U. S. Dept. of Agri. 1 pam.
U. S. Lake Survey Office. 2 pam.
U. S. Naval War Records Office and Library. 4 vols., 5 pam.
Unknown. 1 vol.
Virginia—Dept. of Agri. and Immigration. 1 vol.
Ziino, Sibaldo. 1 pam.

BY PURCHASE.

Rapports de Service. Par E. Dardart et Philippe Dufour. Sténographie. Par Zryd. (Bibliothèque du Conducteur de Travaux Publics.) Paris, H. Dunod et E. Pinat, 1905.

Législation des Eaux. Par Louis Courcelle et E. Dardart. (Bibliothèque du Conducteur de Travaux Publics.) Paris, H. Dunod et E. Pinat, 1905.

Étude sur les Murs de Réservoirs. Par J. B. Krantz. Paris, Dunod, 1870.

Base, Contour and Relief Maps. U. S. Geological Survey. Washington, D. C., 1904.

SUMMARY OF ACCESSIONS.

December 11th, 1905, to January 6th, 1906.

Donations (including 12 duplicates)	125
By purchase	6
Total	131

MEMBERSHIP.

ADDITIONS.

MEMBERS.

		Date of Membership.
ALBER, HERMANN. (Wilson & Alber, Cons. Engrs.), 1216 N. 24th St., Birmingham, Ala		Dec. 6, 1905
BACON, GEORGE MORGAN. 84 R St., Salt Lake City, Utah	Assoc. M. M.	Dec. 3, 1902 Dec. 5, 1905
BALDWIN, ARCHIBALD STUART. Chf. Engr., I. C. R. R., Chicago, Ill		Dec. 6, 1905
BELDEN, HARRY AUSTIN. Care, J. G. White & Co., Inc., Manila, Philippine Islands		Oct. 4, 1905
CARPENTER, CHARLES LINCOLN. Asst. Engr., Isthmian Canal Comm., Bas Obispo, Canal Zone, Panama		Dec. 6, 1905
CLAPP, WILLIAM BILLINGS. 205 So. Fair Oaks Ave., Pasadena, Cal		Dec. 6, 1905
GALLOWAY, JOHN DEBO. Cons. Civ. Engr., Rialto Bldg., San Francisco, Cal		Dec. 6, 1905
GOODRICH, ERNEST PAYSON. Cons. Engr.; Chf. Engr., Bush Cos., 100 Broad St., New York City	Jun. M.	April 3, 1900 Nov. 1, 1905
HUMPHREY, HENRY CYPRIAN. Provincial Superv., Lucena, Tayabas Province, Philippine Islands	Assoc. M. M.	Sept. 4, 1901 Oct. 3, 1905
LEFFINGWELL, FRANK DODGE. Res. Engr., Rapid Transit Subway Constr. Co., 350 Fulton St., Brooklyn, N. Y. (Res., 13 Lexington Ave., Montclair, N. J.)	Assoc. M. M.	Oct. 5, 1898 Dec. 5, 1905
MCCORMACK, EDGAR WALTER. 110 O'Reilly St., Havana, Cuba	Assoc. M. M.	Mar. 6, 1901 Dec. 5, 1905
NEHER, FRANK. Prin. Asst. Engr., Mo. Pac. Ry., 7th and Poplar Sts., St. Louis, Mo		Oct. 4, 1905
NEWTON, ALBERT WILLIAM. 610 Globe-Democrat Bldg., St. Louis, Mo		Dec. 6, 1905
PARKS, OREN ELISHA. Civ. Engr. and Surv., 82 North Elm St., Westfield, Mass		Dec. 6, 1905
WARNER, FRANK CHARLES. U. S. Asst. Engr., Delaware City, Del		Dec. 6, 1905
WOOD, FREDERIC JAMES. Chf. Engr., Boston, Pawtucket & Providence St. Ry. Co.; Cons. Engr., Norfolk & Bristol St. Ry. Co., Foxboro, Mass		Dec. 6, 1905

ASSOCIATE MEMBERS.

ADAMS, RAYMOND EDMOND. Civ. Engr., Quarter Master General's Office, 702 Seventeenth St., N. W., Washington, D. C.	Jun. Assoc. M.	May 1, 1900 Dec. 6, 1905

ASSOCIATE MEMBERS (*Continued*).

		Date of Membership.
ALLEN, EUGENE YORKE. Asst. Res. Engr., Bergen Hill Tunnels, P., N. J. & N. Y. R. R., Weehawken Shaft, P. O. Station No. 1, Hoboken, N. J.		Dec. 6, 1905
BARNES, WALTER ESMOND. 45 Lincoln St., Malden, Mass.	Jun. Assoc. M.	May 6, 1902 Dec. 6, 1905
BUCK, CON MORRISON. Office Engr., A., T. & S. F. Ry., 1006 Garfield Ave., Topeka, Kans.		Dec. 6, 1905
GRADY, JOHN EDWARD. Engr., Bridge Constr., San. Dist. of Chicago, 716 East 50th St., Chicago, Ill.		Dec. 6, 1905
GRAY, JOHN LATHROP. Engr. and Asst. Supt., Paraffine Works of Tide Water Oil Co., Bayonne, N. J.		Dec. 6, 1905
HILDRETH, JOHN LEWIS, JR. Topographical Draftsman, Bureau of Highways, 4 Court Sq., Brooklyn, N. Y.		Dec. 6, 1905
MELIUS, LUDLOW LAWRENCE. Glenmont, N. Y.	Jun. Assoc. M.	May 2, 1899 Dec. 6, 1905
SEVERSON, OSCAR MELVERN. Div. Engr., B. & S. Ry. Co., 992 Ellicott Sq., Buffalo, N. Y.		Dec. 6, 1905
STOCKTON, JOHN. Asst. Engr., C. W. Leavitt, Jr., 15 Cortlandt St., New York City		Dec. 6, 1905
TOOKER, FRANK WESTERVELT. With Chas. W. Leavitt, Jr., 15 Cortlandt St., New York City		Dec. 6, 1905
WIDDICOMBE, ROBERT ALEXANDER. Mgr. and Engr., Steam Fitting Dept., Kroeschell Bros. Co., Chicago, Ill.	Jun. Assoc. M.	April 2, 1901 Dec. 6, 1905
WILSON, WILLIAM EDWARD. Asst. Prof., Civ. Eng., Univ. of Utah; Civ. and Cons. Engr., 14 Eng. Bldg., Univ. of Utah, Salt Lake City, Utah	Jun. Assoc. M.	Jan. 6, 1903 June 7, 1905

JUNIORS.

BLYTHE, LUCIEN HUGUET. 103 Mountain Way, Rutherford, N. J.	Oct. 31, 1905
COOPER, FRANK WESLEY. Laboratory Asst., Structural Materials Lab., U. S. Geological Survey, St. Louis, Mo.	Sept. 5, 1905
CRANE, JOSEPH SPENCER. Asst. Engr. with Wm. P. Field (Res., 179 Washington St.), Newark, N. J.	Dec. 5, 1905
GAY, LEON LINCOLN. Care, U. S. Geological Survey, Minidoka, Idaho	Dec. 5, 1905
HENRY, SMITH TOMPKINS. Assoc. Editor, *The Engineering Record*, 1140 Monadnock Blk., Chicago, Ill.	Sept. 5, 1905
HODGMAN, BURT BRADLEY. 220 Broadway, New York City	Sept. 5, 1905
LEEUW, HENRY. Asst. Engr., Hudson Co., Foot of 15th St., Jersey City (Res., 249 Laurel Ave., Arlington), N. J.	Sept. 5, 1905

JUNIORS (*Continued*).

	Date of Membership.
LIBBEY, JAMES TEMPLETON. Asst. Engr., Erie R. R., P. O. Box 176, Corry, Pa.	Dec. 5, 1905
SPEAR, PHILIP HICHBORN. Draftsman, N. Y. C. & H. R. R. R., 1152 E. Jersey St., Elizabeth, N. J.	Dec. 5, 1905

RESIGNATIONS.

MEMBERS.

	Date of Resignation.
BALBIN, ERNESTO JOAQUIN	December 31st, 1905
FRENCH, ALFRED WILLARD	December 31st, 1905
JARDINE, ALEXANDER WILLIAM	December 31st, 1905

ASSOCIATE MEMBERS.

RAMSEY, EDMUND PAYTON	December 31st, 1905

ASSOCIATES.

BARNES, WILLIAM HENRY	December 31st, 1905
CRAFTS, GEORGE HENRY	December 31st, 1905
PEVERLEY, RALPH	December 31st, 1905

DEATHS.

BLAISDELL, ANTHONY HOUGHTALING. Elected Member, March 3d, 1880; died September 9th, 1905.

MACNAUGHTON, JAMES. Elected Member, May 5th, 1880; died December 29th, 1905.

SNYDER, FRANCIS EDWARD. Elected Member, September 6th, 1905; died December 23d, 1905.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(December 10th, 1905 to January 6th, 1906.)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal,* Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings,* Engrs. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Monadnock Block, Chicago, Ill.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Technology Quarterly,* Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator,* Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(10) *Cassier's Magazine,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News,* New York City, 15c.
(14) *The Engineering Record,* New York City, 12c.
(15) *Railroad Gazette,* New York City, 10c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Street Railway Journal,* New York City. Issues for first Saturday of each month 20c., other issues 10c.
(18) *Railway and Engineering Review,* Chicago, Ill., 10c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 10c.
(21) *Railway Engineer,* London, England, 25c.
(22) *Iron and Coal Trades Review,* London, England, 25c.
(23) *Bulletin,* American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *American Engineer,* New York City, 20c.
(26) *Electrical Review,* London, England.
(27) *Electrical World and Engineer,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, $1.
(29) *Journal,* Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium.
(31) *Annales de l'Assoc. des Ing. Sortis des Ecoles Speciales de Gand,* Brussels, Belgium.
(32) *Memoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Genie Civil,* Paris, France.
(34) *Portefouille Economique des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(36) *La Revue Technique,* Paris, France.
(37) *Review de Mecanique,* Paris, France.
(38) *Revue Generale des Chemins de Fer et des Tramways,* Paris, France.
(39) *Railway Master Mechanic,* Chicago, Ill., 10c.
(40) *Railway Age,* Chicago, Ill., 10c.
(41) *Modern Machinery,* Chicago, Ill., 10c.
(42) *Proceedings,* Am. Inst. Elect. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussees,* Paris, France.
(44) *Journal,* Military Service Institution, Governor's Island, New York Harbor, 50c.
(45) *Mines and Minerals,* Scranton, Pa., 20c.
(46) *Scientific American,* New York City, 8c.
(47) *Mechanical Engineer,* Manchester, England.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.

(54) *Transactions*, Am. Soc. C. E., New York City, $5.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $5.
(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Eng. Soc. W. Pa., 410 Penn Ave., Pittsburg, Pa., 50c.
(59) *Transactions*, Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 20c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 15c.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England.
(70) *Engineering Review*, New York City, 10c.
(71) *Journal*, Iron and Steel Inst., London, England.
(72) *Street Railway Review*, Chicago, 30c.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England.
(78) *Beton und Eisen*, Vienna, Austria.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie-Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(82) *Dinglers Polytechnisches Journal*, Berlin, Germany.
(83) *Progressive Age*, New York City, 15c.

LIST OF ARTICLES.

Bridge.

The Erection of Bridges.* (21) Serial beginning Dec.
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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THE THEORY OF CONTINUOUS COLUMNS.

BY ERNST F. JONSON, ASSOC. M. AM. SOC. C. E.

TO BE PRESENTED MARCH 7TH, 1906.

A column which is continuous through two or more stories differs from a one-story column in that its strength in any story is a function, not merely of its dimensions in that story, but of its dimensions in all stories. A 15-ft. section of a continuous column is evidently stronger when the adjoining sections are 10 ft. long than when they are 15 ft. long, other things being equal. Practically all columns used in buildings of more than one story are continuous.

It is the writer's purpose to develop the exact theory of continuous columns in order to deduce from it a simple method of calculating the effect of eccentric loading, both at the floor level and at an intermediate point.

The development of the equation of the elastic curve, though not new, will be given for the sake of convenience and completeness.

The bending moment in a column is (Fig. 1)

$$M = W\,y \qquad \text{I}$$

where W is the load, and y the distance from the load line to the axis of the column.

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The corresponding stress is

$$s = \frac{W a y}{I} \quad \text{II}$$

where a is the distance to the extreme fiber, and I is the moment of inertia. The corresponding elongation per unit of length is

$$e = \frac{W a y}{E I} \quad \text{III}$$

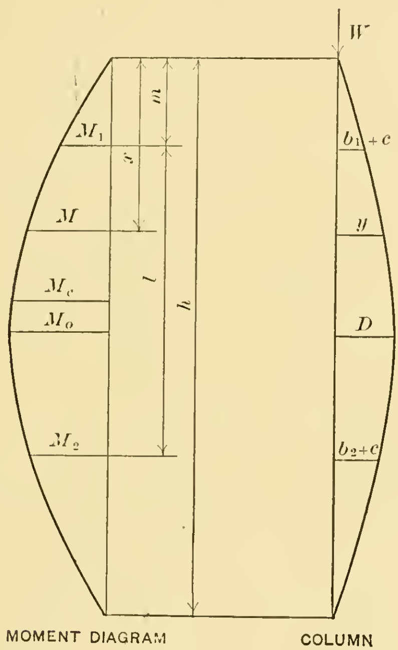

FIG. 1.

where E is the modulus of elasticity. The corresponding second differential may, with sufficient accuracy, be taken as

$$\frac{d^2 y}{d x^2} = -\frac{W y}{E I} \quad \text{IV}$$

Hence the equation of the elastic curve is

$$y = D \sin. x \sqrt{\frac{W}{E I}} \quad \text{V}$$

where $D = y_{max}$.

By multiplying by W we get the equation of the bending moments

$$M = M_o \sin. x \sqrt{\frac{W}{E I}} \qquad \text{VI}$$

where $M_o = M_{max.}$

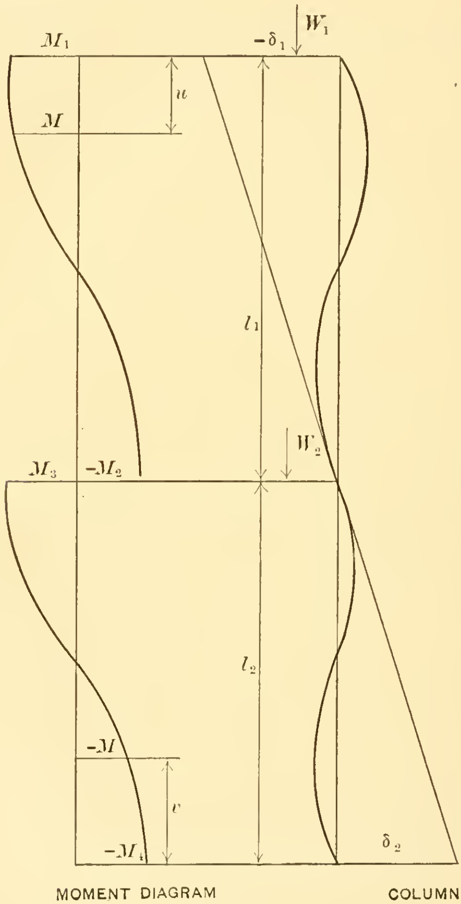

Fig. 2.

The bending of a continuous column in two adjoining stories must fulfil the following condition (Fig. 2):

$$\frac{\delta_1}{l_1} + \frac{\delta_2}{l_2} = 0 \qquad \text{VII}$$

where δ_1 is the deflection of the upper end of the upper column, and δ_2 that of the lower end of the lower one, both measured from the tangent to the elastic curve at the intermediate floor line, and where l_1 is the length of the upper and l_2 that of the lower column.

$$\delta_1 = \frac{1}{E\,I_1}\int_o^{l_1} u\,M\,d\,u \quad \ldots\ldots\ldots\ldots\ldots\text{VIII}$$

$$\delta_2 = \frac{1}{E\,I_2}\int_o^{l_2} v\,M\,d\,v \quad \ldots\ldots\ldots\ldots\ldots\text{IX}$$

where I_1 is the moment of inertia of the upper, and I_2 that of the lower column, and where u is the distance from the top of the upper, and v that from the bottom of the lower column.

Hence Equation VII becomes:

$$\frac{1}{I_1\,l_1}\int_o^{l_1} u\,M\,d\,u + \frac{1}{I_2\,l_2}\int_o^{l_2} v\,M\,d\,v = 0 \ldots\ldots\ldots\text{X}$$

Let $x - u = m$,

$$\int_o^{l} u\,M\,d\,u = M_o\int_o^{m+l} x\ \text{sin.}\ x\sqrt{\frac{W}{E\,I_1}}\,d\,x$$

$$- M_o\int_o^{m} x\ \text{sin.}\ x\sqrt{\frac{W}{E\,I_1}}\,d\,x - M_o\,m\int_o^{m+l}\text{sin.}\ x\sqrt{\frac{W}{E\,I}}\,d\,x$$

$$+ M_o\,m\int_o^{m}\text{sin.}\ x\sqrt{\frac{W}{E\,I}}\,d\,x,$$

hence

$$\int_o^{l} u\,M\,d\,u = M_o\,\frac{E\,I}{W}\Bigg[\text{sin.}\ (m+l)\sqrt{\frac{W}{E\,I}}$$

$$-(m+l)\sqrt{\frac{W}{E\,I}}\ \text{cos.}\ (m+l)\sqrt{\frac{W}{E\,I}}\Bigg]$$

$$- M_o\,\frac{E\,I}{W}\Bigg[\text{sin.}\ m\sqrt{\frac{W}{E\,I}} - m\sqrt{\frac{W}{E\,I}}\ \text{cos.}\ m\sqrt{\frac{W}{E\,I}}\Bigg]$$

$$- M_o\,m\sqrt{\frac{E\,I}{W}}\Bigg[1 - \text{cos.}\ (m+l)\sqrt{\frac{W}{E\,I}}\Bigg]$$

$$+ M_o\,m\sqrt{\frac{E\,I}{W}}\Bigg[1 - \text{cos.}\ m\sqrt{\frac{W}{E\,I}}\Bigg].$$

$$\int_o^{l} u\,M\,d\,u = M_o\,\frac{E\,I}{W}\Bigg[\text{sin.}\ (m+l)\sqrt{\frac{W}{E\,I}}$$

$$- l\sqrt{\frac{W}{E\,I}}\ \text{cos.}\ (m+l)\sqrt{\frac{W}{E\,I}} - \text{sin.}\ m\sqrt{\frac{W}{E\,I}}\Bigg] \ldots\ldots\text{XI}$$

From the Moment Equation VI, we find

$$M_1 = M_o \sin. m \sqrt{\frac{W}{E I}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{XII}$$

$$M_2 = M_o \sin. (m + l) \sqrt{\frac{W}{E I}} \quad \ldots\ldots\ldots\ldots \text{XIII}$$

$$\sin. m \sqrt{\frac{W}{E I}} = \sin. (m + l) \sqrt{\frac{W}{E I}} \cos. l \sqrt{\frac{W}{E I}}$$

$$- \cos. (m + l) \sqrt{\frac{W}{E I}} \sin. l \sqrt{\frac{W}{E I}}.$$

$$\frac{\sin. (m + l) \sqrt{\frac{W}{E I}} \cos. l \sqrt{\frac{W}{E I}} - \sin. m \sqrt{\frac{W}{E I}}}{\sin. l \sqrt{\frac{W}{E I}}} = \cos. (m + l) \sqrt{\frac{W}{E I}}.$$

$$\frac{M_2 \cos. l \sqrt{\frac{W}{E I}} - M_1}{\sin. l \sqrt{\frac{W}{E I}}} = M_o \cos. (m + l) \sqrt{\frac{W}{E I}} \quad \ldots\ldots \text{XIV}$$

Introducing these values into Equation XI, we get:

$$\int_o^l u\, M\, d u = \frac{E I}{W} \left[M_2 - l \sqrt{\frac{W}{E I}} \frac{M_2 \cos. l \sqrt{\frac{W}{E I}} - M_1}{\sin. l \sqrt{\frac{W}{E I}}} - M_1 \right]$$

$$\int_o^l u\, M\, d u = \frac{E I}{W} \left[M_1 \left(\frac{l \sqrt{\frac{W}{E I}}}{\sin. l \sqrt{\frac{W}{E I}}} - 1 \right) \right.$$

$$\left. + M_2 \left(1 - \frac{l \sqrt{\frac{W}{E I}}}{\tan. l \sqrt{\frac{W}{E I}}} \right) \right] \quad \ldots\ldots\ldots\ldots \text{XV}$$

Introducing this value into Equation X, we get:

$$\frac{M_1}{W_1\, l_1} \left[\frac{l_1 \sqrt{\frac{W_1}{E\, I_1}}}{\sin. l_1 \sqrt{\frac{W_1}{E\, I_1}}} - 1 \right] + \frac{M_2}{W_1\, l_1} \left[1 - \frac{l_1 \sqrt{\frac{W_1}{E\, I_1}}}{\tan. l_1 \sqrt{\frac{W_1}{E\, I_1}}} \right]$$

$$+ \frac{M_3}{W_2 l_2} \left[1 - \frac{l_2 \sqrt{\frac{W_2}{E I_2}}}{\tan. l_2 \sqrt{\frac{W_2}{E I_2}}} \right]$$

$$+ \frac{M_4}{W_2 l_2} \left[\frac{l_2 \sqrt{\frac{W_2}{E I_2}}}{\sin. l_2 \sqrt{\frac{W_2}{E I_2}}} - 1 \right] = 0 \ldots\ldots \text{XVI}$$

This equation is a "theorem of four moments" for continuous columns.

We also know that

$$M_2 - M_3 + M_e = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{XVII}$$

where M_e is the moment due to eccentricity of loading, the left-hand side being the positive side.

By applying these two equations to each story, from top to bottom of a continuous column, the two end moments of each section of the column are found in terms of the lower end moment of the column below, and by working back from bottom to top, introducing the values of the lower end moment, the absolute values will be found.

From Equations XII and XIII we find the maximum moment

$$M_o = \frac{\sqrt{M^2_1 + M^2_2 - 2 M_1 M_2 \cos. l \sqrt{\frac{W}{E I}}}}{\sin. l \sqrt{\frac{W}{E I}}} \ldots\ldots\ldots \text{XVIII}$$

From Equation VI we get the distance, h, between two points in which the elastic curve, if prolonged, would intersect the load line.

$$h = \pi \sqrt{\frac{E I}{W}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{XIX}$$

This is Euler's formula for the length of an ideal, centrally-loaded column. The distances, m and $m + l$, from the end of the ideal column to the two ends of the actual column will be, according to Equation VI,

$$m = \frac{h}{\pi} \text{ arc sin. } \frac{M_1}{M} \ldots\ldots\ldots\ldots\ldots\ldots \text{XX}$$

$$m + l = \frac{h}{\pi} \text{ arc sin. } \frac{M_2}{M_o} \ldots\ldots\ldots\ldots\ldots\ldots \text{XXI}$$

From these values of m and $m + l$ it will be seen whether or not the maximum moment falls within the actual length of the column.

Let M_c be the moment at the middle of the column. According to Equation VI

$$M_c = M_o \sin. \left(m + \frac{l}{2}\right) \sqrt{\frac{W}{E\,I}}$$

$$M_c = M_o \left(\sin. m \sqrt{\frac{W}{E\,I}} \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}} + \cos. m \sqrt{\frac{W}{E\,I}} \sin. \frac{l}{2} \sqrt{\frac{W}{E\,I}}\right) \ldots\ldots \text{XXII}$$

According to Equation XIII

$$M_2 = M_o \left(\sin. m \sqrt{\frac{W}{E\,I}} \cos. l \sqrt{\frac{W}{E\,I}} + \cos. m \sqrt{\frac{W}{E\,I}} \sin. l \sqrt{\frac{W}{E\,I}}\right) =$$

$$M_o \left(\sin. m \sqrt{\frac{W}{E\,I}} \cos.^2 \frac{l}{2} \sqrt{\frac{W}{E\,I}} - \sin. m \sqrt{\frac{W}{E\,I}} \sin.^2 \frac{l}{2} \sqrt{\frac{W}{E\,I}} + 2 \cos. m \sqrt{\frac{W}{E\,I}} \sin. \frac{l}{2} \sqrt{\frac{W}{E\,I}} \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}\right)$$

hence

$$M_o \cos. m \sqrt{\frac{W}{E\,I}} \sin. \frac{l}{2} \sqrt{\frac{W}{E\,I}} = \frac{M_2}{2 \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}}$$

$$- M_c \frac{\sin. m \sqrt{\frac{W}{E\,I}} \cos.^2 \frac{l}{2} \sqrt{\frac{W}{E\,I}}}{2 \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}} + M_o \frac{\sin. m \sqrt{\frac{W}{E\,I}} \sin.^2 \frac{l}{2} \sqrt{\frac{W}{E\,I}}}{2 \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}}$$

Inserting this value in Equation XXIII, we have

$$M_c = \frac{M_2}{2 \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}} + M_o \frac{1}{2} \sin. m \sqrt{\frac{W}{E\,I}} \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}$$

$$+ M_o \frac{\sin. m \sqrt{\frac{W}{E\,I}} \sin.^2 \frac{l}{2} \sqrt{\frac{W}{E\,I}}}{2 \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}}$$

$$M_c = \frac{M_2 + M_o \sin. m \sqrt{\frac{W}{E\,I}}}{2 \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}}$$

Substituting M_1 for $M_o \sin. m \sqrt{\frac{W}{E\,I}}$, as per Equation XII, we have

$$M_c = \frac{M_1 + M_2}{2 \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}} \qquad \text{XXIV}$$

Hence it is seen that when, as is usually the case in buildings, here is but little variation in the function, $\frac{l}{2} \sqrt{\frac{W}{E\,I}}$, for any two adjoining stories, M_c may be taken as proportional to the average of M_1 and M_2. This implies that we may, without great error, neglect the curvature of the moment diagram when the theorem of four moments becomes:*

$$\frac{l_1}{I_1} (M_1 + 2\ M_2) + \frac{l_2}{I_2} (2\ M_3 + M_4) = 0 \qquad \text{XXV}$$

Any possible irregularity in a column, such as a bend, an unsymmetrical distribution of the material, or a variation in the modulus of elasticity, may be reduced to an equivalent eccentricity of loading. This equivalent eccentricity, which we will call c, may be deduced from experiments by means of Equation V, if we substitute c for y, $\cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}$ for $\sin. x \sqrt{\frac{W}{E\,I}}$ (provided that the specimen is supplied with round or knife-edge bearings, as it should be, in order to give reliable results), and insert the actual value of the deflection, which is

$$D = \frac{I\,(s - w)}{a\,W} \qquad \text{XXVI}$$

hence

$$c = \frac{I\,(s - w)}{a\,W} \cos. \frac{l}{2} \sqrt{\frac{W}{E\,I}}$$

or

$$c = \frac{r^2\,(s - w)}{a\,w} \cos. \frac{l}{2\,r} \sqrt{\frac{w}{E}} \qquad \text{XXVII}$$

where r is the radius of gyration, and w the average unit stress.

For immediate practical use, c may be deduced from the particular building law or specification under which the work is to be executed. Thus the writer gets, from the New York building law, a c for steel columns equal to $\frac{1}{16}$ in. per foot of column length, and

* See the writer's paper "The Theory of Frameworks with Rectangular Panels, and its Application to Buildings which have to Resist Wind," in *Transactions*, Am. Soc. C. E., Vol. LV, p. 413.

for cast-iron columns it becomes one-third of the radius of gyration. These values are round figures, which, in the writer's opinion, harmonize with the spirit of the law.

The actual values vary with length, radius of gyration, and distance to extreme fiber. This value for c must be added to every actual eccentricity, so that Equation XVII becomes

$$M_2 - M_3 + M_e + W_1 c_1 + W_2 c_2 = 0 \dots\dots\dots\dots \text{XXVIII}$$

When there is considerable difference between the numerical values of M_1 and M_2, M_0 must be calculated by Equation XIX, but when they are about equal and of the same sign the simpler formula, Equation XXIV, may be used, as there will then be but little difference between M_0 and M_c. When M_1 and M_2 are of about the same numerical value, but of opposite sign, as is usually the case in buildings, M_0 falls outside the actual length of the column and, therefore, need not be considered, the length being seldom greater than half the distance between the maximum moments. The New York building law allows a maximum length of 64% of that distance.

From Equation XXV it is seen that when there is not much difference in the functions, $\frac{l}{I}$ and M, of the various stories, we may assume

$$M_1 = -M_2 = M_3 = -M_4$$

We then get the following simple formula which will cover most cases of eccentric loading occurring in buildings:

$$-M_2 = M_3 = W\left(c + \frac{b}{2}\right) \dots\dots\dots\dots \text{XXIX}$$

where b is the eccentricity of the resultant load. The only condition to be fulfilled by each of these moments being

$$M_e \overline{\overline{<}} \frac{I}{a}(p - w) \dots\dots\dots\dots \text{XXX}$$

p being the safe compressive stress of the material.

When there are moments in two directions at right angles to each other the best way is to calculate the column in the two directions separately, and then add the resulting unit stresses. Equation XXX then becomes

$$\frac{M_{ex}\,a_x}{I_x} + \frac{M_{ey}\,a_y}{I_y} \overline{\overline{<}} p - w \dots\dots\dots\dots \text{XXXI}$$

If an eccentric load be applied to a column at a point inter-

mediate between two fixed points or floor levels, Equation VII becomes (Fig. 3)

$$\frac{\delta_1}{l_1} + \frac{\delta_2}{l_2} = \frac{\alpha_1 - \alpha_2}{l_1} + \frac{\alpha_3 - \alpha_2}{l_2} \ldots\ldots\ldots\ldots \text{XXXII}$$

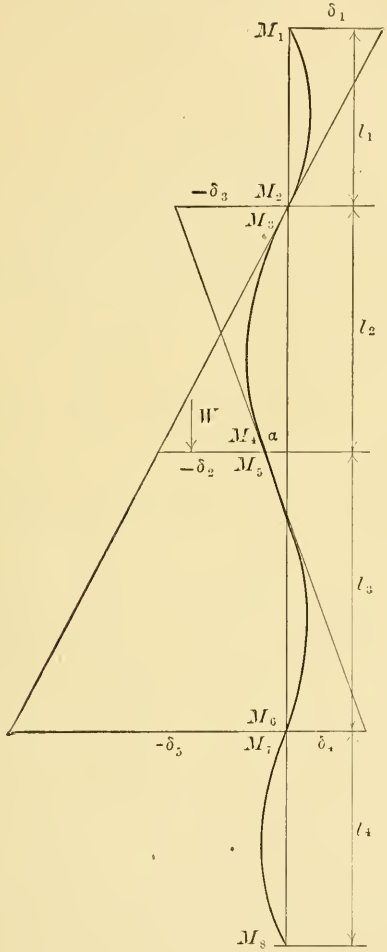

FIG. 3.

where α_1, α_2 and α_3 are the deflections of three successive points at which external forces are applied. We also need an equation covering three sections or column lengths

$$\frac{\delta_1}{l_1} + \frac{\delta_5}{l_2 + l_3} = 0$$

where δ_5 is the deflection of the lower end of the lower section measured from the tangent to the elastic curve at the upper end of the middle section. Hence

$$\delta_5 = \frac{\delta_2 (l_2 + l_3)}{l_2} + \frac{\delta_3\, l_3}{l_2} + \delta_4$$

where δ_3 and δ_4 correspond in the middle and bottom sections to δ_1 and δ_2 of the top and middle sections. Hence

$$\frac{\delta_1}{l_1} + \frac{\delta_2}{l_2} + \frac{\delta_3\, l_3}{l_2\,(l_2 + l_3)} + \frac{\delta_4}{l_2 + l_3} = 0 \ldots\ldots \text{XXXIII}$$

If we now insert the various values of δ in Equations XXXII and XXXIII, we get the final equations. The exact values are given by Equations VIII, IX and XV, but, for practical purposes, the approximate value used in Equation XXV will be sufficiently accurate. The equations then become:

$$\frac{l_1}{I_1}\,(M_1 + 2\,M_2) + \frac{l_2}{I_2}\,(2\,M_3 + M_4)$$
$$= 6\,E\left(\frac{\alpha_1 - \alpha_2}{l_1} + \frac{\alpha_3 - \alpha_2}{l_2}\right) \ldots\ldots \text{XXXIV}$$

$$\frac{l_1}{I_1}\,(M_1 + 2\,M_2) + \frac{l_2}{I_2}\,(2\,M_3 + M_4) + \frac{l_3}{I_2\,(l_2 + l_3)}\,(M_3 + 2\,M_4)$$
$$+ \frac{l_3}{I_3\,(l_2 + l_3)}\,(2\,M_5 + M_6) = 0 \ldots\ldots \text{XXXV}$$

By means of Equations XXVI and XXXIV we find the various moments, as previously explained; not the absolute value, however, but a value expressed in terms of α, which is the deflection of the unsupported point. The value of α is found by applying Equation XXXV to the two sections above the unsupported point and the one below it. The equation may also be applied to the two sections below and the one above, if it be reversed, that is, if M_1 and M_6, M_2 and M_5, M_3 and M_4, l_1 and l_3, exchange places.

If the column is only two sections long, one above and one below the unsupported point, and pin-connected at both ends, the above formulas cannot be used, for the problem is then a statically determined one. When the bottom end of a column is fixed, the foundation may be considered as a section of the column, the moment of inertia of which is infinite.

As to centrally-loaded columns, the writer would suggest the

following modification of the present practice, namely, that, instead of proportioning a column according to its own length alone, it would be better to take the length as one-half the distance from the middle of the story below to that of the story above. By the present practice the long columns are made relatively stronger than the short ones.

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PAPERS AND DISCUSSIONS.

NEW FACTS ABOUT EYE-BARS.

By Theodore Cooper, M. Am. Soc. C. E.

To be Presented March 21st, 1906.

When, in the course of professional practice, new facts are discovered which either broaden or contradict previously accepted beliefs, it is a professional duty to present the results and deductions, after careful examination, for the common benefit of our fellow workers.

"Prove all things; hold fast that which is good."

In the execution of the superstructure of the Quebec Bridge, with its 1 800-ft. channel span, the great magnitude of the members, the high working strains, and other features of the work have demanded careful study of many points, which, in ordinary bridges, could be and have been overlooked or neglected as of small importance.

This paper will be confined to the new facts developed in regard to eye-bars.

It was a surprise and a cause of much anxiety to the writer to discover how defective was our knowledge of the eye-bar.

As a general rule, we have failed to recognize that the real elongation of an eye-bar is from out to out of pin-holes, and not

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

from center to center of pins. We have carefully determined its elongation and elastic limit over a certain length of the parallel bar, and then accepted this determination as equally true when applied to the whole bar. We have assumed that a set of bars carefully bored to an exact length would all pull to an equal strain, as long as the elastic limit measured on the body of the bar was not exceeded. All these beliefs and assumptions are incorrect.

In ordinary bridges this has not been a matter of much importance, owing to the low unit strains and the small change in the deformations.

Description of the Investigation.

In August, 1904, the manufacture of the eye-bars for the anchor arms of the Quebec Bridge was well under way when the question arose as to what clearance should be allowed between the pins and pin-holes. The eye-bars forming the tension members were 15 in. in width, from $1\frac{1}{4}$ to $2\frac{1}{16}$ in. in thickness, and of lengths from 50 to 58 ft. The pins, except in a few special cases, were 12 in. in diameter and from 8 to 10 ft. in length. The maximum joint has 58 bars on one pin.

After discussion, it was decided that the clearance necessary for the purposes of erection should not be less than $\frac{1}{32}$ nor more than $\frac{1}{16}$ in., the latter being about the proportion given to ordinary bridge pins.

As the maximum working strains are higher than in usual practice, being about 21 000 lb. per sq. in. in tension, the intensity and distribution of the local pressures from the pin to the eye of the bar became important.

The writer, after a little consideration of the problem, realized that, while the local pressure must be very great, and the pin-holes must deform elliptically, at least elastically and probably permanently under the proposed working strains, the solution could only be obtained by experiment. He then devised a method of measuring the elongation of the bar from out to out of pins, while the bar was strained to varying amounts in the testing machine.

Four bars with different clearances were prepared and tested to 12 000, 16 000, 20 000, 24 000 and 28 000 lb. per sq. in.

While the result of these tests (Nos. 646, 647, 648 and 649 of Table 2) was not fully satisfactory, owing to the crudeness of the

hurriedly made appliance, and the difficulty of reading the fine measurements in the limited space, they showed that the bars, from out to out of pins, began to elongate permanently at 12 000 lb., and that the elongation increased with each increase of strain. The amount of the pin clearance did not modify the results especially.

Was this deformation, even at low strains, a peculiarity of this "make" of bars, or had it been observed in other tests? Looking up old records, the writer found in his abstract of tests made at the St. Louis Bridge in 1872, that of 58 eye-bars put to the proof test of 18 000 lb. per sq. in., 5 showed a permanent elongation of the pin-holes of $\frac{1}{16}$ in., 52 of them $\frac{1}{32}$ in., and 1 of them $\frac{1}{64}$ in. These were iron bars.

In the Watertown "Tests of Metals" for 1883, there were found tests on 6 steel eye-bars, where the permanent elongation between pin centers at different strains is noted. They are abstracted in Table 1.

TABLE 1.—BARS, $6\frac{1}{2}$ BY 1 IN.; PINS, 5 IN.; EXCESS, 40 PER CENT. AT SIDES; END SECTION, 86 PER CENT.

No. of Bar.	Stretch of Bar, out to out of Pin-Holes, in Inches.						Ultimate Strength.
	10 000	20 000	25 000	30 000	35 000	40 000	
4582	0.015	0.020	.....	.....	0.090	2.72	67 800
4583	0.010	0.020	0.025	0.040	0.060	2.63	64 000
4584	0.015	0.020	0.025	0.040	0.050	....	65 000
4585	0.020	0.025	0.030	0.045	0.070	....	65 850
4586	0.020	0.030	0.030	0.045	0.175	....	64 400
4587	0.010	0.015	0.025	0.030	0.060	....	68 290

It became evident, therefore, that the stretch of the eyes of eye-bars was not peculiar to the present "make" of bars, but had always occurred.

As it was important to push the construction of the work, it was decided to give the 12-in. pins $\frac{3}{64}$ in. clearance, and, for the anchor arms then under construction, to add an extra allowance of $\frac{1}{32}$ in. to the elastic elongation of each eye-bar for camber determinations; and should the further and fuller tests, then determined upon, show this to be too much or too little, the correction could be made in the cantilever arms.

The importance of the permanent stretch of the eyes, as affecting the structure in other directions, was not overlooked, but the data so far obtained were too few to furnish any definite conclusions.

Preparations were made for fuller tests, and, to eliminate the difficulties of the first method of measuring the stretch and also to get the action of each eye independently, the following method was adopted: Measure each bar from out to out of eyes, and calliper each eye longitudinally and transversely before putting it in the testing machine. Then, after straining the bar to 12 000, 16 000, 20 000, etc., lb. per sq. in., remove it from the machine and repeat the measurements.

In preparing the bars for test, it was determined to get from bars already made such a selection as would give a wide range in "heat numbers," "thicknesses," "proportions of the head," and "pin clearances." Some of the bars were specially bored to change the proportions of the head and the pin clearances, and two bars with visible flaws in the head were selected.

This selection covers Bars Nos. 705 to 718. The later bars are those which have since then been selected from time to time for the usual proof tests.

In Tables 2 and 3 all the important data of the tests so far made have been entered.

The records have been given as recorded. It will be noticed that the tape measurements from out to out of eyes, while they agree reasonably well with the sum of the elongations of the two eyes in most cases, differ in other cases. This may be partly due to errors of measurement and partly due to the measurements being taken on one side of the bar only; which, in the case of the bar being warped by the strain, would not give the exact length.

It should also be noted that in taking out and replacing the bar, if it did not get the exact position it first occupied on the pin, there would be an additional elongation of the hole before it got its proper bearings.

A number of the bars were additionally tested by trying to maintain a constant strain for several hours, to determine the effect of time. There was an increase of stretch, but it is believed to be at least partially due to the difficulties of holding a constant pressure on the machine for a long time.

TABLE 2.

No. of bar.	Thickness, in inches.	Head.	Excess, percentage.	Pin clearance, in inches.	Stretch of Pin-Holes, in Inches, at Varying Strains per Square Inch. 12 000	16 000	20 000	24 000	28 000	Rupture.	Stretch, Out to Out of Pin-Holes, by Tape. 24 000	Ultimate strength of bar.	Heat number.	Specimen Tests of Heat. Tensile strength.	Elongation. percentage.	Remarks.
646	1¼	A	44	0.033	0.037	0.052	0.055	0.113	0.138	3.6	....	60 900	14 069	63 970	27	These were the first bars te·ted. The results given are only approximate, as the gauge appears to have slipped, especially in 648 and 649.
		B	44	0.035						3.6				65 180	27.5	*Untrustworthy.
647	1¼	A	54	0.033	0.010	0.011	0.020	0.021	0.020	3.2	....	61 220	14 069	66 800	30	
		B	44	0.031						3.3						
648	2	A	48	0.072	0.070*	0.069*	0.089*	0.103*	0.088	3.4	....	56 200	15 281	60 720	26	
		B	57	0.069					0.038*	3.1				64 730	28	
649	2	A	39	0.057	0.012*	0.060*	......	......	0.114*	5.1	....	57 000	15 281	65 320	25	
		B	50	0.060						3.4						
705	$1\frac{11}{16}$	A	52	0.073	0.014	0.017	0.033	0.081	....	1.7	$\frac{3}{8}$	51 230*	16 672	60 300	31	Broke at flaw in head B.
		B	49	0.073	0.010	0.029	0.072	0.205	....	2.4*				60 300	32	704 cut from same bar, broke at 57 190 and pin holes elongated 3.5 and 3 in.
														62 560	30	
706	$1\frac{15}{16}$	A	48	0.064	0.009	0.044	0.112	0.266	....	3.6	$\frac{39}{64}$	57 730	7 666	61 380	29.5	Cut from same bar.
		B	48	0.073	0.000	0.035	0.084	0.207	....	3.5				61 880	27	
707	$1\frac{15}{16}$	A	42	0.082	0.006	0.013	0.045	0.148	....	3.1	$\frac{11}{32}$	55 160	7 666	66 090	26	
		B	46	0.075	0.010	0.028	0.055	0.135	....	3.0						
708	$1\frac{13}{16}$	A	40	0.050	0.004	0.006	0.012	0.032	....	2.5	$\frac{1}{8}$	59 450	13 250	65 980	24	" " "
		B	39	0.048	0.000	0.017	0.044	0.059	....	3.7				66 220	25	
709	$1\frac{13}{16}$	A	31	0.084	0.000	0.016	0.043	0.106	....	3.0	$\frac{1}{4}$	58 340	13 250	66 760	26	
		B	43	0.075	0.016	0.020	0.035	0.105	....	3.3						
710	$1\frac{9}{16}$	A	53	0.072	0.005	0.005	0.009	0.015	....	1.7	$\frac{3}{64}$	60 230	14 069	63 970	27	
		B	41	0.075	0.003	0.012	0.025	0.042	....	3.2				65 180	27.5	
711	$1\frac{9}{16}$	A	49	0.051	0.000	0.000	0.000	0.031	....	2.1	0	58 960	14 069	66 800	30	
		B	48	0.070	0.000	0.000	0.000	0.045	....	2.6						
712	$1\frac{9}{16}$	A	46	0.034	0.002	0.009	0.014	0.023	....	2.5	$\frac{3}{64}$	63 210	14 069			" " " as 716.
		B	40	0.037	0.000	0.007	0.012	0.027	....	3.8						
713	$1\frac{3}{8}$	A	58	0.051	0.000	0.002	0.003	0.012	....	2.0	$\frac{1}{32}$	65 220	4 853	64 380	23	" " "
		B	54	0.054	0.000	0.000	0.005	0.006	....	1.4				65 200	23	
714	$1\frac{3}{8}$	A	55	0.064	0.008	0.008	0.014	0.037	....	2.1	$\frac{3}{64}$	65 220	4 853	67 800	24.5	
		B	51	0.063	0.000	0.002	0.006	0.024	....	2.3						
716	$1\frac{9}{16}$	A	69	0.038	0.006	0.006	0.006	0.007	....	1.1	$\frac{1}{64}$	57 800†	14 069			† Broke at flaw in head B. Cut from same bar as 712.
		B	61	0.043	0.000	0.000	0.003	0.011	....	....						
717	$1\frac{9}{16}$	A	49	0.018	....	......	0.010	0.028	....	3.2	$\frac{3}{64}$	64 360	14 074	61 780	29	Cut from same bar.
		B	42	0.060	....	......	0.008	0.029	....	3.1				65 440	25.5	
718	$1\frac{9}{16}$	A	59	0.064	....	......	0.006	0.019	....	2.5	$\frac{1}{32}$	64 870	14 074	67 000	28.5	
		B	63	0.043	....	......	0.002	0.009	....	2.4						

TABLE 3.

No. of bar.	Thickness, in inches.	Head.	Excess. percentage.	Pin clearance, in inches.	Stretch of Pin-Holes, in Inches, at Varying Strains per Square Inch. 16 000	18 000	20 000	24 000	26 000	35 200	Stretch, Out to Out of Pin-Holes, by Tape. 24 000	Ultimate strength of bar.	Heat number.	Specimen Tests of Heat. Tensile strength.	Elongation. percentage.	Remarks.
760	$1\frac{1}{4}$	A	62	0.049	0.017	0.017	0.018	0.069	0.090	0.614	$\frac{7}{64}$	......	14 215			
		B	66	0.054	0.019	0.020	0.024	0.048	0.072	0.666						
761	$1\frac{1}{4}$	A	56	0.074	0.009	0.013	0.030	0.085	0.136	0.726	$\frac{9}{64}$	......	14 215	61 180	24.5	
		B	59	0.053	0.000	0.009	0.015	0.037	0.066	0.649				64 020	26.5	Cut from same bar.
762	$1\frac{1}{4}$	A	52	0.065	0.019	0.032	0.039	0.090	0.138	0.942	$\frac{9}{64}$	......	14 215	65 640	24	
		B	59	0.050	0.009	0.012	0.016	0.039	0.055	0.720						
763	$1\frac{1}{4}$	A	55	0.054	0.009	0.008	0.016	0.033	0.056	0.821	$\frac{3}{32}$	......	14 215			
		B	51	0.052	0.009	0.015	0.022	0.062	0.105	0.740						
										Ultimate.						
798	2	A	47	0.047	......	......	0.017	0.035	......	3.0	$\frac{7}{64}$	56 430	12 937	60 150	26.5	
		B	45	0.054	......	......	0.000	0.020	......	3.8				61 860	29	" " " "
799	2	A	46	0.044	......	......	0.011	0.042	......	3.2	$\frac{7}{64}$	56 740	12 937	65 180	26.5	
		B	41	0.047	......	......	0.016	0.029	......	2.8						
800	$1\frac{7}{16}$	A	47	0.063	......	......	0.028	0.065	......	3.7	$\frac{1}{8}$	57 780	9 919	66 320	24	
		B	50	0.052	......	......	0.038	0.057	......	3.8				67 220	23.5	" " " "
801	$1\frac{7}{16}$	A	49	0.050	......	......	0.032	0.056	......	3.6	$\frac{7}{64}$	57 080	9 919	69 220	23	
		B	47	0.049	......	......	0.024	0.055	......	3.7						
802	$1\frac{9}{16}$	A	43	0.049	......	......	0.022	0.061	......	4.4	$\frac{3}{16}$	53 800	16 099	66 580	26	
		B	47	0.053	......	......	0.027	0.069	......	3.9				66 670	27	14 in. pins.
803	$1\frac{9}{16}$	A	46	0.050	......	......	0.022	0.050	......	4.3	$\frac{1}{8}$	54 300	16 099	68 050	23	Cut from same bar.
		B	47	0.051	......	......	0.023	0.049	......	4.8						
804	2	A	44	0.056	......	......	0.029	0.058	......	3.9	$\frac{7}{64}$	59 500	16 093	64 900	25	
		B	14	0.046	......	......	0.019	0.034	......	4.3				65 230	25.5	A pins 12 in.—B pins 14 in.
807	2	A	46	0.053	......	......	0.018	0.036	......	3.0	$\frac{7}{64}$	58 800	16 093	65 500	26	Cut from same bar.
		B	47	0.033	......	......	0.040	0.079	......	4.0						
808	$1\frac{3}{4}$	A	41	0.056	......	......	0.008	0.043	......	3.9	$\frac{5}{64}$	58 240	12 891	63 180	29	
		B	42	0.062	......	......	0.012	0.036	......	2.8				64 260	27	Cut from same bar.
809	$1\frac{3}{4}$	A	44	0.054	......	......	0.008	0.030	......	3.2	$\frac{5}{64}$	57 670	12 891	67 179	24	
		B	48	0.049	......	......	0.008	0.034	......	3.1						
822	$1\frac{15}{16}$	A	48	0.052	......	......	0.010	0.019	......	3.5	$\frac{3}{64}$	60 940	8 690	....	....	
		B	48	0.057	......	......	0.012	0.025	......	2.9						" " " "
823	$1\frac{15}{16}$	A	51	0.044	......	......	0.017	0.022	......	3.5	$\frac{5}{64}$	60 440	8 690	....	....	
		B	47	0.070	......	......	0.021	0.034	......	4.8						
759	$2\frac{1}{8}$	A	58.6	0.046	0.000	0.007	0.007	0.012	0.025	....	$\frac{7}{64}$	......	......	....	....	
		B	Gross.	0.026	0.016	0.029	0.040	0.050	0.064	....						Riveted links, made of plates.
758	$2\frac{1}{8}$	A	58.6	0.060	0.000	0.007	0.010	0.022	0.027	....	$\frac{3}{32}$	......	......	....	....	
		B	Gross.	0.058	0.000	0.013	0.017	0.028	0.037	....						

To illustrate the method of the testing, one detailed test is here given:

February 8th, 1905. Test No. 711. Bar, 15 by $1\frac{9}{16}$ in. Heat number, 14 069.

Head A.	Head B.	
21.46 by 1.64 in.	21.60 by 1.62 in.	Elastic limit, 32 850 lb.
Excess, 49.4%.	Excess, 48.5%.	Ultimate strength, 58 960 lb.
Original area of bar, 23.56 in.		Fracture, 40% silky, 60% fine granular, half cupped.
Fractured area of bar, 13.66 in.		

Diameter of testing machine pin, 11.98 in.

TABLE 4.—DETAILED OBSERVATIONS.

Strain per square inch, in pounds.	Diameters of Pin Holes. A.	Diameters of Pin Holes. B.	Out to out of pin-holes.	10 ft. on body of bar.
0	*T* 12.032 *L* 12.031	*T* 12.050 *L* 12.050	$15\text{–}9\frac{9}{16}$	10
12 000	*T* 12.032 *L* 12.031	*T* 12.050 *L* 12.050	"	"
16 000	*T* 12.032 *L* 12.031	*T* 12.050 *L* 12.053	"	"
20 000	*T* 12.032 *L* 12.031	*T* 12.050 *L* 12.060	"	"
24 000	*T* 12.031 *L* 12.040	*T* 12.048 *L* 12.075	"	"
24 000 after first 2 hr.	*T* 12.031 *L* 12.040	*T* 12.047 *L* 12.084	$15\text{–}9\frac{9}{32}$	"
24 000 after second 2 hr.	*T* 12.030 *L* 12.064	*T* 12.047 *L* 12.095	$15\text{–}9\frac{5}{8}$	$10\text{–}0\frac{1}{32}$
24 000 after third 2 hr.	*T* 12.030 *L* 12.064	*T* 12.047 *L* 12.095	"	"
After rupture	*T* 12.00 *L* 14.12	*T* 12.00 *L* 14.66	$18\text{–}7\frac{1}{2}$	12–28

After a number of tests had been made, from 12 000 to 24 000 lb., it was found that the important data could be obtained with less frequent removals, and thus save much time and labor. Therefore, with the exception of some special tests, the records afterward were taken only at 20 000 and 24 000 lb.

In testing two connecting links, Nos. 758 and 759, made of plates riveted together, four bars, Nos. 760-763, were used to make connection with the testing machine. The data for these members were extended over a wider range, but not carried to rupture, as the un-

supported eyes of the links began to buckle in advance of the pins at the higher strain.

On plotting the results, using the stretch of the holes after rupture as the upper limit, it was found that the stretch of the eyes at various strains per square inch followed a regular curve, differing for the different bars, but all having one general form, Fig. 1.

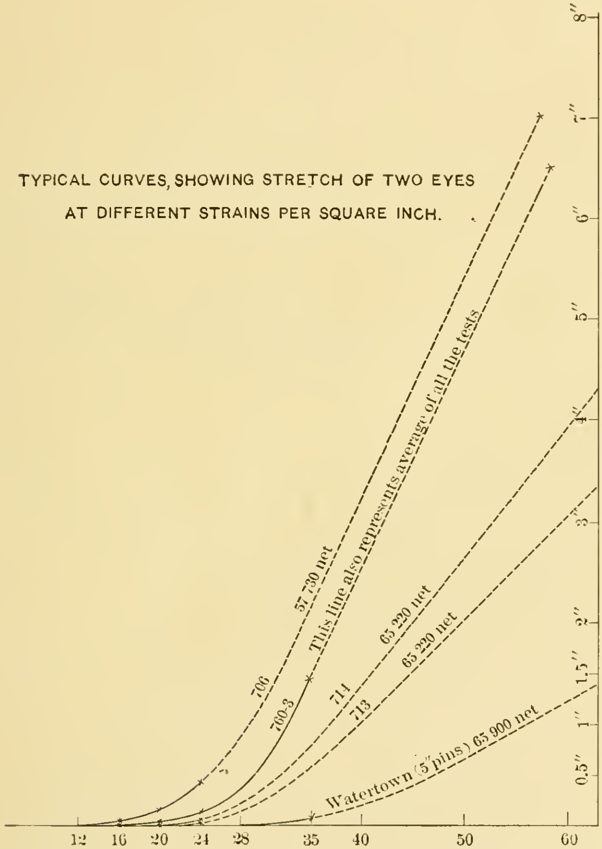


Fig. 1.

In order to save confusion by plotting each particular curve, the bars have been arranged in eight different classes, according to their stretch (covering both eyes). The average of each class is plotted in Fig. 2 on a larger scale than Fig. 1.

In addition to the previous observations, a number of the heads were scribed with fine lines, longitudinally and transversely, dividing the heads into spaces 2 in. square. After the rupture of the bars, these lines were traced and plotted, for comparison with the plots

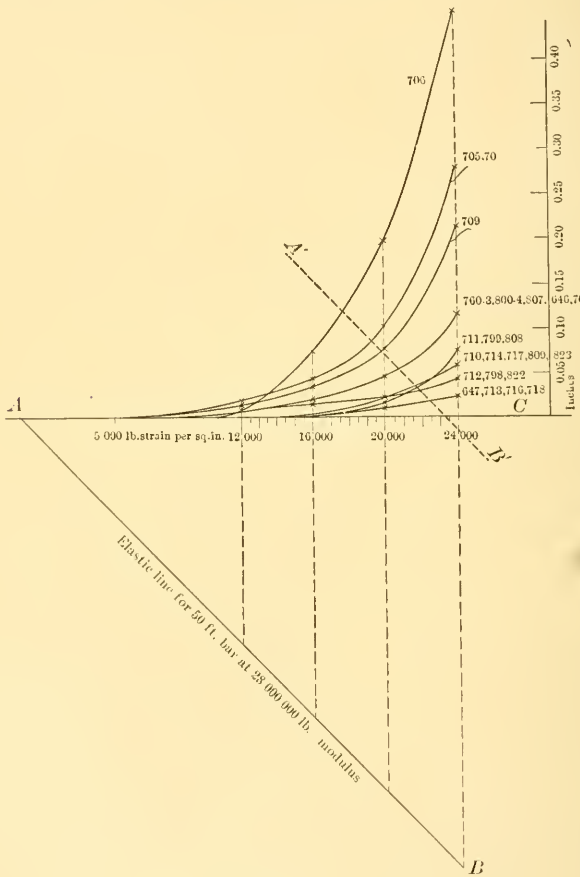


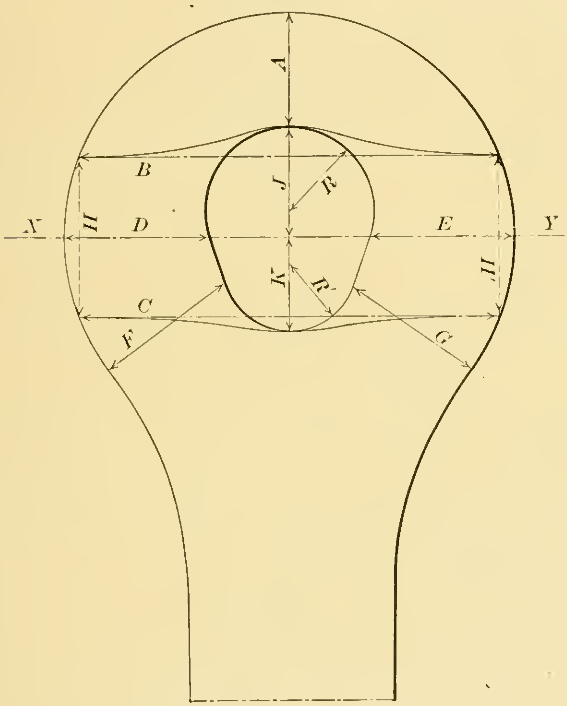
FIG. 2.

of the lines on the original bars, to determine the relative flow of the metal in different parts of the head.

It would be difficult to reproduce these tracings on a small scale. In Fig. 3 the principal and important changes from the original dimensions are shown, and the values are given in Table 5.

The line, X Y, is the transverse line through the center of the original pin-hole. The curved lines tangent to the elongated pin-hole are the forms taken by the straight lines tangent to the original hole before straining the bars. The upper part of the pin-hole is held to the form and diameter of the pin and has elongated more than the lower half. The diameter of the lower half has decreased by the

DEFORMATION OF HEADS OF EYE-BARS UNDER RUPTURE.



X-Y is line through center of original pin-hole

FIG. 3.

transverse closing in of the material under the pull. At the top of the pin the metal of the head is decreased in depth by the compression. The transverse dimensions across the eye and neck are reduced by the flow of the metal under tension. It is interesting to note that with these proportioned heads the distances, H and H', on the outsides of the heads opposite the pins, have elongated very little or not at all, which would indicate that the periphery of the

heads at these points had not been strained much more than the elastic strength of the metal, though the bar had been strained to rupture. It should be noted that in some of the 10 and 8-in. bars this distance on one side has been decreased, indicating a compressive distortion on one side. As, in two cases, it was as much as $\frac{1}{16}$ and $\frac{5}{64}$ in., it would not appear to be due to errors in measurement, the portion of the metal most severely taxed being that portion of the intrados of the eye lying between the horizontal and curved lines at the top of the pin.

The head, No. 708 *A*, pulled unequally in the neck, one side, *G*, decreasing $1\frac{1}{4}$ in., while the other side, *F*, only decreased $\frac{3}{8}$ in., showing softer metal at one side than the other.

The difference in the pulling of the heads of No. 710 may be partly due to the fact that No. 710 *A* was thicker, having an excess of 53%, while No. 710 *B* had only 44 per cent.

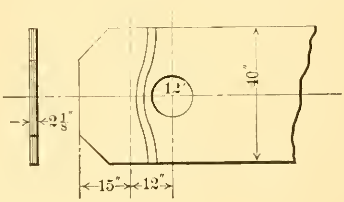


Fig. 4.

Similar observations were made upon some 8 and 10-in. bars, with like results.

The heads of the riveted links, not strained to rupture, showed a different action.

All the transverse lines below the center of the pin and those more than 12 in. above the center of the pin moved in a parallel direction, while those from the center of the pin to about 12 in. above took a curved form as shown in Fig. 4.

This was due to the absence of any neck below the pin, and to the greater stiffness of the material above the pin to resist bending.

Consideration of the Results.

As far as relates to the original purpose of the first tests, *viz.*, to determine the effect of the pin clearances, the tests give no definite answer. The different pin clearances vary from 0.031 to 0.084 in.

TABLE 5.

Bars.	Pins.	No. of Head.	Diameter of Head.	Deformation on Lines. (See Fig. 3.)											
				A	B	C	D	E	F	G	H	H'	J	K	R'
	Inches.		Inches.	In Sixty-fourths of an Inch.									Inches.	Inches.	Inches.
15 inches........	12	708A	33	—16	—48	—88	—40	—40	—24	—80	+0+	+0+	$+1\frac{5}{8}$	$+\frac{7}{8}$	5
		710A	33.7	—16	—12	—32	— 8	—16	—16	—20	+0+	+0+	$+1\frac{1}{16}$	$+\frac{11}{16}$	$5\frac{1}{4}$
		710B	33.4	—48	—64	—68	—48	—48	—64	—48	+0+	+0+	$+2\frac{1}{8}$	$+1\frac{1}{8}$	$4\frac{3}{4}$
		713A	35	—32	— 8	—20	— 0+	— 0+	— 0+	— 0+	+0+	+0+	$+1\frac{3}{8}$	$+\frac{5}{8}$	$5\frac{3}{8}$
		713B	34.8	— 7	—20	—24	—12	—12	—10	— 0+	+4	+4	$+\frac{13}{16}$	$+\frac{11}{16}$	$5\frac{1}{4}$
		714A	34.9	—28	—32	—48	—24	—24	— 8	—16	0	0	$+1\frac{9}{16}$	$+\frac{13}{16}$	$5\frac{3}{16}$
		714B	34.75	—28	—16	—44	—16	—16	— 0	—16	0	0	$+1\frac{1}{2}$	+1	$5\frac{3}{16}$
10 inches........	$7\frac{31}{32}$	$1\frac{3}{8}A$	$22\frac{5}{16}$	—22	—19	—14	— 4	—44	— 0	—16	+3	—4	$+1\frac{13}{64}$	$+\frac{15}{64}$	$3\frac{21}{64}$
		$1\frac{3}{8}B$	$22\frac{7}{64}$	—36	—64	— 56	—14	—24	—18	— 3	+3	+3	$+1\frac{21}{32}$	$+\frac{45}{64}$	$3\frac{17}{64}$
		$1\frac{1}{2}A$	$22\frac{25}{32}$	—28	—27	—45	—12	—23	—18	— 3	0	0	$+1\frac{53}{64}$	$+\frac{19}{32}$	$3\frac{23}{64}$
		$1\frac{1}{2}B$	$22\frac{13}{16}$	—28	—19	—14	—18	—19	— 9	— 8	+3	—5	$+1\frac{43}{64}$	$+\frac{17}{32}$	$3\frac{1}{4}$
8 inches.........	$6\frac{5}{16}$	$1\frac{3}{8}A$	$18\frac{17}{32}$	—12	—18	—18	— 8	— 9	— 3	— 3	0	0	$+1\frac{13}{64}$	$+\frac{46}{64}$	$2\frac{31}{32}$
		$1\frac{3}{8}B$	$18\frac{9}{32}$	—14	— 8	—16	—10	—10	—11	— 3	—1	+2	$+1\frac{21}{32}$	$+\frac{45}{64}$	3
		$1\frac{3}{4}A$	$18\frac{1}{2}$	—16	—20	—20	—14	—19	— 5	— 5	+2	+2	$+1\frac{1}{32}$	$+\frac{1}{2}$	$2\frac{23}{32}$
		$1\frac{3}{4}B$	$18\frac{3}{8}$	—18	—34	—27	—12	— 8	—13	— 2	—1	+6	+1	$+\frac{5}{16}$	$2\frac{15}{16}$

— Decrease. + Increase.

While, no doubt, the pin clearance has some influence on the stretch of the eyes, it is hidden in the far greater influence of other factors. The eyes would undoubtedly elongate permanently were the pins fitted perfectly tight.

In like manner, the influence of the percentages of excess of the head is indeterminate. Bar No. 709, with excesses of 31 and 43%, gave the same result at each eye. The eight eyes of the four bars Nos. 760-3, which were made from one mill bar, gave the following elongations at 24 000 lb.:

Head.	Excess.	Elongation.
763 *B*	51%	0.062
762 *A*	52	0.090
763 *A*	55	0.033
761 *A*	56	0.085
761 *B*	59	0.037
762 *B*	59	0.039
760 *A*	62	0.060
760 *B*	66	0.048

In these and other tests, however, there appears to be an influence due to the excess of material at the end of the eyes, which would indicate that for the best results this excess should be limited.

In the Watertown tests quoted, and in the riveted links Nos. 758-9, the frontal section in the first being 86%, and in the second 119%, of the body of the piece, the material at the end of the eye tended to buckle instead of stretching, as is the case with smaller percentages in the end of the head. The study of the present tests leads the writer to believe that a great stretch of the eyes before rupture, heretofore considered as showing a tough and tenacious material, is no more desirable than a tendency to buckle in front of the pin. The best proportions of head to resist the elongation of the eyes under the working strains cannot be decided by the present tests. It is thought probable that, for circular heads, 50% excess across the eye, thus making the end section 75% of the bar, would be a favorable proportion.

In a general examination of the tests it will be seen that the bars of the higher tensile strengths gave the better results.

Before the tests had gone very far, it was decided that the tensile

strength should be advanced, the percentages of the heads increased, and the pin clearances for the bridge bars limited to $\frac{3}{64}$ in.

While the pin clearances, excess of heads, tensile strength, and thickness of the bars, as affecting the tensile qualities, undoubtedly have some influence upon the stretch of the eyes, they do not give a full explanation.

The original bridge bars, being more than 50 ft. long, were cut in half and additional eyes made, so as to make two test bars. In one case, above mentioned, four test bars were made from one bridge bar.

The four heads of Bars Nos. 706 and 707 (same original bar) give stretches, at 24 000 lb. per sq. in., ranging from 0.135 to 0.266 in., or about as 1 to 2. That a long bar (more than 50 ft.), would differ in quality at the two extremes, does not explain the difference, for, on any assumption as to which two heads were made at the adjoining cut ends, there is still a minimum difference of stretch of 0.059 in.

Similarly, for Nos. 708 and 709 there is a range from 0.032 to 0.106 in. with a minimum difference for any two adjoining heads of C.074 in.

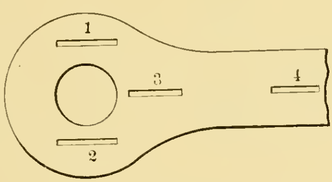


Fig. 5.

It will be noticed that the bars first tested gave the worst results (and it was fortunate that this was the case, for otherwise the necessity for a fuller series of tests might not have been recognized), presumptive evidence that more care had been taken for the later bars. The manufacturers and the inspectors assured the writer, however, that no change had been made in any of the processes of the manufacture of the later bars. It was then decided to cut samples from the two worst heads, Nos. 705 *B* and 706 *A*, and from one of the best, No. 711*A*, for nicked fractures and for tensile test, to see what the difference would be.

Samples Nos. 1 and 2 were cut from each side of the head, No. 3 from the neck, and No. 4 from the body, of the bar, as shown by Fig. 5.

The nicked fractures of the samples cut from Nos. 705 and 706 showed the same uniform fine granular structure with a clear bright luster. No difference could be detected between the several samples.

For No. 711, the samples showed a slightly coarser grain, with a suspicion of yellowish tinge in the samples from the head and neck.

These samples were examined, while fresh, by practical steel workers and experts. All agreed that they indicated nothing which would explain the different action of these heads.

The tensile tests (samples unannealed) are shown in Table 6.

TABLE 6.—TENSILE TESTS.

Head. No.	No. of Sample.	Ultimate Strength.	Elongation in 8 in.	Reduction of area.	Fracture.
705 *B*................	1	56 730	26%	52.8%	Silky, cup.
	2	60 870	25	53.8	" angular.
	3	59 260	26	56.5	" cup.
	4	61 680	28.2	56.6	" angular.
706 *A*................	1	62 400	23	51.9	Silky, angular.
	2	62 750	24.5	48.8	" "
	3	60 060	25.5	59.1	" "
	4	75 760	9.5	50.6	" "
711 *A*...........	1	64 800	23.2	55.7	Silky, angular.
	2	64 960	19.5	51.7	" ½ cup.
	3	69 900	26	50.2	" angular.
	4	80 760	8	46.7	" cup.
Another sample cut from the body of bar No. 711, annealed.	5	60 430	25.8	55.8	Silky.

NOTE: As Bar No. 705 broke at a flaw in the head, *B*, the full tensile strength of the bar was not developed.

An examination of the tests for the riveted links, Nos. 758 and 759, shows that, even here, where there were no heat treatments, either of forging or annealing, the stretch of the eyes varied, the two eyes of No. 759 varying at each strain, and at 24 000 lb. the difference was 0.038 in.

An inspection of Figs. 1 and 2 shows that each class of bars, after giving a steadily increasing stretch, up to a certain point for each class, then begins to yield more and more rapidly, the bars of the higher tensile strength, as a rule, and presumably the harder

bars, resist this breaking down up to a higher point. The unequal pulling of the metal in different heads and in different parts of the same head, as shown in Table 5 and Fig. 3, shows that the metal is not homogeneous, but is softer in some bars and in different parts of the same bar. It is probable that the breaking down of the metal in front of the pin unequally is a large factor in the problem.

It is undoubtedly a great mistake to seek a soft and ductile eye-bar by using either low tensile material or softening processes.

To get the best results, we must either use steel of a higher grade, or else stretch the eyes longitudinally, cold, before the final boring to exact length, or do both. The writer believes that, with proper appliances, eye-bars can be made, which will not stretch in the eyes within the maximum working strains, without greatly increasing their cost.

As the tests here recorded have extended over a year's time, and every effort has been made to have them fairly represent the bars as manufactured from time to time, it is believed that the actual bridge bars will be better than those tested, the pin clearances and proportions of the heads being better and the tensile strength somewhat higher.

These tests do not take any account of the elastic elongation of the eyes, which no doubt occurs, but would probably be small and constant for the different bars.

Practical Consideration and Application of the Results of the Tests.

Upon the development of the fact that eye-bars were not perfectly elastic even at low working strains, and took an increasing permanent stretch with increasing loads, it became a grave question as to "How will a series of such bars pull together?"

If two or more bars with different curves of stretch are strained to a fixed amount on the same pins, the parallelism of which is assured, the total elongation, $A\ C$, in Fig. 6, would be equal, but the permanent elongations, $B\ C$, being different, the elastic elongations, $A\ B$, must have a like difference.

The elastic elongations, $A\ B + A'\ B'$, corresponding to the total load, must be divided between the two bars proportionately to $A\ B$ and $A'\ B'$. The difference of strain on the two bars will depend

upon the ratio of the difference of permanent stretches, $B\ B'$, to the average elastic elongation, $A\ B + A'\ B' \div 2$.

The permanent stretch, at any working strain, being independent of the length of the bar, while the elastic elongation is proportionate to the length of the bar, the difference of strain in two such bars will be less as the lengths of the bars are greater.

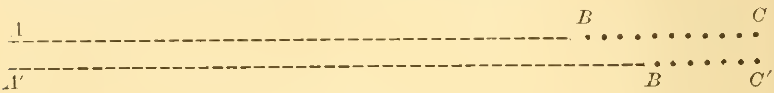


Fig. 6.

In Fig. 1 the line, $A\ B$, is the elastic line for a bar 50 ft. long. The vertical ordinates between $A\ B$ and $A\ C$ give the elastic elongations of a bar of this length for any strain per square inch. The vertical distances from $A\ C$ to the curve of any bar gives its permanent stretch for each strain. Any line, $A'\ B'$, drawn through the curves of stretch of the several bars parallel to $A\ B$, will give, at the points of crossing, the strain on each bar for that condition of loading. The total elongations, being between two parallel lines, must be equal at these points.

For bars of other lengths, the elastic line must be changed to suit each particular length.

Although it is believed that the actual bridge bars are better than the bars tested, it will be assumed that the various bars shown in Fig. 2 cover the extremes and represent the variety of bars to be used.

By drawing any line, $A'\ B'$, parallel to the elastic line for a bar 50 ft. long, and taking off the strain on each set of bars, we can readily get the average strain for all the bars and the limits of the variation, for example:

Bar.	No.	Strain.	Sum.
1	706	17 600	17 600
2	705–7	19 200	38 400
1	709	19 750	19 700
12	760, etc.	20 600	247 200
3	711 "	21 600	64 800
5	710 "	21 600	108 000
3	712 "	21 800	65 400
4	713 "	22 450	89 800
—		———	———
31		Average, 21 000	650 900

Which indicates that for bars of this kind, when pulled together on pins held parallel, for an average working strain of 21 000 lb. per sq. in., the softest bar will have only 17 600 lb. per sq. in., or about 84% of the average strain; and the hardest bar will have 22 450 lb. per sq. in., or about 107% of the average—strains not disproportionate to the capabilities of the different kinds of bars.

For the longer bars, up to 58 ft.—50 ft. being the minimum length—the difference in strain will be still less.

For bars of short lengths, under high working strains, the difference in strain becomes very great, which renders the use of such bars very undesirable. It will be seen that, for much lower working strains and short bars, the bars will be subject to a like variation of strain, with long bars and the higher strains.

It is very sure, therefore, that, when using high working strains, as are required for structures of great magnitude, long bars only must be used, if this stretch of the eyes cannot be overcome.

Camber.

For the working strain of 21 000 lb. per sq. in., it was found that a full set of bars of this kind would take a permanent elongation of about $\frac{1}{16}$ in., or $\frac{1}{32}$ in., for each eye, and this amount was provided for in all camber determinations. Further, it is thought that the probable error at the center of the channel span will not be more than $1\frac{1}{2}$ in. either way, an amount of no importance.

There are other features, connected with the action of such bars, which have been considered and provided for, but they do not come within the scope of the present paper.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
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PAPERS AND DISCUSSIONS.

A NEW GRAVING DOCK AT NAGASAKI, JAPAN.

Discussion.*

By Messrs. Charles M. Jacobs, R. C. Hollyday and L. J. Le Conte.

Mr. Jacobs.

Charles M. Jacobs, M. Am. Soc. C. E.—The Japanese engineers are to be congratulated on avoiding one of the great errors made in the early period of graving dock building, namely, in the width of the entrance. They have avoided this, which has detracted from the usefulness of a large number of docks in Europe; but the speaker suggests that they have committed another error in not dividing the dock by a central caisson or by gates. For example, the dock could be subdivided into two equal lengths. Looking at the photograph, Plate CII,† it will be observed that a number of torpedo boats are on the blocks, the result being that the repairs to all must be completed, at least temporarily, before floating out. The length of ordinary trading vessels on the coasts of Japan and China averages from 320 to 360 ft. Assuming that a seriously damaged vessel arrives in the dock for repairs, the entire dock is immediately closed for other work until the damaged vessel is ready for refloating. If there had been a midway caisson, or gates, the seriously damaged vessel could remain at the upper end while the outer section of the dock would remain free for docking vessels requiring light repairs, painting, cleansing, etc. Therefore the commercial possibilities and utility of the dock, with comparatively small extra expense, would

* Continued from December, 1905, *Proceedings*. See October, 1905, *Proceedings* for paper on this subject by Naoji Shiraishi, M. Am. Soc. C. E.

† *Proceedings*, Am. Soc. C. E., for October, 1905.

have been enhanced considerably by its subdivision, and it would still be available when required for ships of the largest tonnage. Mr. Jacobs.

Another point in the paper is surprising, and that is the expense of the adoption of cut stone overlying the concrete, which, according to the figures of costs given in the table, was about seven times the cost of concrete. It is beyond all question that the utilization of concrete alone for graving dock construction has long passed the experimental stage, and is assuming enormous proportions in all classes of engineering structures. The experience of the speaker, in building a short time ago two large graving docks in South Wales entirely of concrete, has proven that concrete is absolutely satisfactory in maintenance, notwithstanding that the walls of this Welsh dock had to pass through most treacherous quicksand before reaching a gravel foundation, and that the total rise and fall of the tides was 32 ft. Therefore it seems to be an unnecessary extravagance to place a granite lining on top of the concrete when the latter is just as efficient, if care be taken to enrich the outer surfaces of the concrete on the sides, altars, steps, and coping.

The speaker hopes he may also be pardoned for criticising the assertion in regard to the non-use of mechanical power, notwithstanding the statement as to the cheapness of labor in Japan. Take the cost of the dock lately completed by the speaker in South Wales, of nearly the same dimensions as the Nagasaki Dock. The contract price for the Welsh dock, built entirely of concrete, and having central gates, was \$445 000, whereas the cost of the Japanese dock, notwithstanding the cheap labor, was \$700 000. The cost of labor in Great Britain is very much higher than in Japan, and, with the utilization of mechanical power, the costs have proven more economical.

Another point in the paper to which attention has been called is the utilization of puzzolana mixed with the concrete to increase its imperviousness. The great difficulty in all graving dock construction is to obtain water-tightness, due to intermittent stresses on the structure. Particulars of this important feature have been omitted. If the author could give some records of filtration through the walls and invert it would add very much to the interest of the paper.

R. C. Hollyday, M. Am. Soc. C. E.—The speaker, having been engaged in the construction of dry docks for the Navy Department during the past 10 years, believes that it may be of interest to state what has been done. Mr. Hollyday.

As Engineer-in-Charge, the speaker was connected with the construction of the timber dry dock at the Puget Sound Naval Station, completed about 1896, at a cost of \$700 000. This dock rests on piles, with a concrete foundation under the bottom and for 7 ft. on the sides under the altars. The entrance of the dock, for 75 ft.,

Mr. Hollyday. including, of course, both gate seats, is of concrete faced with cut stone. The pumps are operated by steam engines. It is the only timber dock constructed by the Navy Department which has proved serviceable or at all satisfactory. At about the same time that this dock was being built, a timber dock was being built at the Port Royal Naval Station, South Carolina. The latter has never been satisfactory, and has seldom been used; to-day, it is scarcely in condition to be used at all, and certainly not for any first-class battleship. It is slightly smaller than the Puget Sound dock. At about the same time, Dry Dock No. 3 was being constructed at the New York Navy Yard. This dock is, approximately, of the same size as the Puget Sound dock, being about 625 ft. on the floor and 668 ft. in length over all. It was operated originally by steam-driven pumps, but a new pumping plant has recently been installed for this dock and Dry Dock No. 2, one plant being connected for both, and operated by three motor-driven 45-in. centrifugal pumps, each having an average capacity of 50 000 gal. per min. The original cost of this dock was $794 000. A great deal of trouble was encountered during its construction, and a large part of it was reconstructed almost immediately after it was completed. The cost of reconstruction was about $300 000, and even then the dock was in a very unsatisfactory condition. Three years ago the dock had to be put out of commission for about 6 months during repairs. The repairs were of a rather novel character, and pertained to the foundations near the entrance, principally in cutting off a large stream of water which entered the dock from the harbor. This flow of water undermined the material laid under the dock and under its sides to such an extent that it was likely to give way at this point. A great deal of the underlying material was quicksand, which caused most of the trouble in the construction and in its maintenance. It is hoped that this dock may be kept in commission until the completion of Dry Dock No. 4, which is now being constructed at the New York Navy Yard, and then rebuilt and made a masonry dock of concrete, lined with cut stone.

Dry Dock No. 2 at the Mare Island Navy Yard, California, which the speaker was next connected with, as Engineer-in-Charge, is now under construction. It was originally intended to be of timber, but, by Act of Congress, it was ordered that it be of stone.

The next dock with which the speaker was connected, as Engineer-in-Charge, was one of masonry at the Boston Navy Yard. This was completed during 1905. It is a masonry dock, of concrete faced with granite, and is 788 ft. in length. It was authorized at the same time as the masonry dock for the Portsmouth, New Hampshire, Navy Yard, which, by the way, is located across the river from Portsmouth, at Kittery, Me. This and the Portsmouth dock

Mr. Hollyday.

are approximately the same size; both are operated by electrically-driven pumps, and have all the accessories, in the nature of winches and capstans for handling vessels, all electrically operated. They are also provided with pneumatic pipe lines for furnishing power for work on ships in docks; with conduits for electric wires for power and for lighting; and with water piping, with outlets at frequent intervals, for attaching hose for washing down the dock and for whatever purposes water may be needed there. They are docks of the highest type yet completed, and have all the modern appliances for handling ships expeditiously, economically, and efficiently, and for doing necessary work on ships after they are docked.

The dock with which the speaker is at present connected, in the capacity of Engineer-in-Charge of construction, is Dry Dock No. 4, at the New York Navy Yard. This is to be of concrete, and little or no cut stone will be used for facing, in fact, none except at the gate seats. The dock is to be 516 ft. long on the floor by 78 ft., 542 ft. on the top by 130 ft., and will have a depth of 31 ft. of water over the sill. The contract for this dock was made during 1905, and 42 months are allowed for completion. It is to be operated by electrically-driven centrifugal pumps. Owing to the limit of cost for this dock, and also the limited space available at the New York Navy Yard, it is not possible to make it as long as the other docks which have recently been authorized by Congress for the Navy Department. It is large enough to dock the largest battleship, and it is not probable that a battleship will ever be built so large as not to be able to enter it. It is also large enough to take in the largest cruisers yet built, but it is not at all improbable that, in the future, cruisers may be built for the Navy which will be too large to enter this dock. However, Dry Dock No. 3 is long enough to take in any cruiser which is likely to be built.

Masonry docks are also being built at the navy yards at League Island, Charleston, and Norfolk. All are large docks, and of the highest type, having all the modern appliances of the Portsmouth and Boston docks.

Mr. Jacobs states that probably the Japanese made a mistake in going to the expense of facing the dock at Nagasaki with cut stone. In the speaker's opinion, this was a very important feature, and along the right lines. There are two ways of looking at this question: one from the commercial standpoint, and the other from a Government or engineering standpoint. From a commercial standpoint, it is necessary to design and build a dock which will pay a reasonable interest on the investment. A commercial concern, in building a dock, may not have the capital to invest, and, in any event, cannot afford, from a business standpoint, to make an investment which is known in advance will not yield a reasonable

Mr. Hollyday. interest. The Government and engineering standpoints are almost the same. In each case it is desirable to build the best structure possible at anything less than what might be called a prohibitive cost. From the Government standpoint, durability and efficiency are the essential features, and to have a structure which will always be available for use whenever it may be needed—a structure where extensive repairs are not continually needed. When repairs are being made it is frequently necessary to put the dock out of commission; at such a time it would not be available for use, and that might be the very time it might be needed most.

At the New York Navy Yard there are three docks, and a fourth one is under construction. Dry Dock No. 1, of masonry, with granite facing; Dry Dock No. 2, originally of timber, completed in 1890, reconstructed and built of masonry with concrete facing and metal strips for protecting the edge of the altar steps and coping, the reconstruction being completed in 1902; Dry Dock No. 3, of timber, completed in 1897. The cost of repairs on the bodies of these docks from January, 1903, to date, has been, approximately: Dry Dock No. 1, nothing; Dry Dock No. 2, $950; Dry Dock No. 3, $17 200. Although Dry Dock No. 2 has only been completed about three years, the concrete walls have begun to give trouble. Water finds its way through the walls at different points, and is first shown by the action of frost during the winter, the cracks become enlarged, the concrete is gradually spawled off the face, and the disintegration goes on. Repairs have been made to the facing, but these repairs are only regarded as temporary, patching work, and it is not believed possible to repair the dock so that there will not be future trouble of the same kind. This trouble would not have occurred had the dock been faced with cut stone, and this illustrates very clearly the point which the speaker makes in regard to facing the dock at Nagasaki with cut stone rather than concrete.

Originally, the Government built only masonry docks faced with cut stone, and the stone was frequently cut with much greater nicety than was necessary, but this was rather a matter of detail and cost. Dry Dock No. 1 at the Boston Navy Yard is of stone, and was completed in 1833. Dry Dock No. 1 at the Mare Island Navy Yard was completed in 1891; it was built by day labor, and, owing to interruption of appropriations, the work was not continuous and it took some 12 years to build it. Dry Dock No. 1 at the New York Navy Yard is of stone, and was completed about 1846. Dry Dock No. 1 at Norfolk is of stone, and was completed in 1827. These four stone docks—the first docks built by the Navy Department—may be said to be in practically as perfect a state of preservation as ever. During all these years practically no money has been spent on the body of these docks in the way of repairs, whereas, all the

Mr. Hollyday.

timber dry docks built by the Navy Department and completed within the last 10 or 12 years, with the exception of the Puget Sound dry dock, have required very extensive repairs, and one of them is practically useless to-day. This ought to show pretty conclusively that the only course open to the Government is to build the most substantial structure possible.

This Society may not be aware of the fact, but an Informal Discussion on "Dry Docks—Stone *vs.* Wood,"* had considerable to do with influencing the policy of the Government in regard to the kind of docks to be built. In 1898 Congress authorized the construction of four large dry docks—at Portsmouth, Boston, League Island, and Mare Island Navy Yards—the Portsmouth and Boston docks were to be of masonry, and the League Island and Mare Island docks were to be of timber. At that time opinion in the Navy Department was not unanimous as to the kind of dock which should be built, and there was a strong interest favoring the construction of timber docks. Congress had made up its mind to build at least two timber dry docks, against the recommendation of the Secretary of the Navy, as advised by Rear Admiral M. T. Endicott, M. Am. Soc. C. E., Chief of the Bureau of Yards and Docks, and Rear Admiral G. W. Melville, Hon. M. Am. Soc. C. E., Chief of the Bureau of Steam Engineering. There was a great deal of discussion as to the type of dock to be built, but Congress did not change its decision. Under the authority vested in him, the Secretary of the Navy proceeded to have plans prepared, advertise, and make contracts, for the construction of two stone docks at the Portsmouth and Boston Navy Yards. He also took the necessary steps for building the two timber docks authorized for the League Island and Mare Island Navy Yards, with a view to having them changed to stone docks by Act of Congress later. During this time, the subject of stone *versus* timber docks was brought up and discussed before this Society, and it was found that those who took part in the discussion strongly favored the stone dry dock. When Congress met, during the following year, members of the Naval Committee were informed that the question of stone *versus* timber dry docks had been discussed very thoroughly before this Society, and that the consensus of opinion of those who had discussed the subject was strongly in favor of stone dry docks; that Congress had made a mistake in adhering to the policy of building timber dry docks, and that it was not too late to rectify the error. The speaker was informed, by one of the most prominent members of the Naval Committee, that the discussion before this Society had as much weight as, or more than, any other one thing in influencing Congress in reversing its policy and author-

* *Transactions*, Am. Soc. C. E., Vol. XLI. p. 554.

Mr. Hollyday. izing the construction of the League Island and Mare Island docks of masonry.

It may not appear to members of this Society that this is a matter of very much importance, but to the speaker it is, in that this discussion could change the policy of the Government from a wrong principle to a right one.

Although this Society disclaims all responsibility for the facts and opinions advanced in any of its publications, its policy has always been and now is to discuss engineering subjects from an engineering standpoint. It has been against entangling alliances, of all kinds and descriptions. There has never been any suspicion of its furthering the interests of any man or set of men, and by this wise policy it has come about that its opinion is turned to as one of absolute disinterestedness.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The author was especially fortunate in having had available such a good site for a dock. Such favorable locations are very rare. Undoubtedly, the entire base of the dock should rest on solid rock, wherever possible.

The depth of water at the site of the coffer-dam, 50 ft. at high water, to bed-rock, although somewhat excessive, is often exceeded, and special precautions have to be taken on account of the great pressure on the dam when the enclosure is pumped out. A rock-filled dam is generally adopted in such cases, but practice differs with regard to the puddle core.

A semi-circular dam of small stone, properly filled with good clayey material to choke the voids on the outer slope often fulfils all requirements where that slope is not exposed to heavy weather. Where labor is so cheap and the rock excavation is seamy, as in this case, it is certainly wise to conduct the work with hand labor throughout, as the likelihood of injuring the foundation rock forbids all blasting near the lining work.

The writer is pleased with the proportions of the concrete, and with the addition of puzzolana to make impervious work, which was very desirable; he is somewhat surprised, however, to note the heavy expense for cutstone facing—$167 500—the average rate being $15 per cu. yd. in place. In these days of economical construction it is not at all clear why the altars, at least, could not have been made of high-grade concrete, if not the entire dock lining.

The writer knows that comparisons are always odious, because conditions are so different in different cases, but thinks that some broad general comparisons would not be out of place.

The new dock at Mare Island, California, now under construction, is on a pile foundation throughout, the dock foundation alone calling for 14 000 piles. The site is quite unfavorable. The dock will be 734 by 120 by 35 ft., and will have an aggregate capacity

of 114 180 cu. yd. When completed it will probably cost in the neighborhood of $1 800 000, making a capacity rate of $15.76 per cu. yd. Mr. Le Conte.

The Hamilton Graving Dock, at Malta, completed in 1893-94, was founded entirely on rock, but the formation was full of fissures, some of them 8 in. wide, and much water had to be contended with. Besides the cost of the dock proper, there were also included the cost of a factory for repairs, a 160-ton hydraulic crane, and shops and stores for repair and storage of gun mountings, etc. All this brought the cost up to $1 023 163. The dock is 558 by 126 by 44 ft., and has an aggregate capacity of 114 576 cu. yd., which makes the cost $8.93 per cu. yd. Next comes the author's dock, having a cost of $5.56 per cu. yd. of capacity, and finally, the new Hunter's Point Dry Dock, San Francisco, Cal., completed in 1903. This dock is founded throughout on rock. The lining is almost entirely of concrete, excepting the masonry piers at the entrance. The dimensions are 750 by 122 by 36 ft. = 122 000 cu. yd. Of this, 100 000 cu. yd. was rock excavation. The cost of engineering construction alone has been only $488 000, making the cost $4 per cu. yd. The altars on both sides of the dock are entirely of high-grade concrete.

This stands out in strong contrast with the Mare Island Dock, above mentioned, which is situated only a few miles further inland.

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PAPERS AND DISCUSSIONS.

THE INSPECTION OF TREATMENT FOR THE PROTECTION OF TIMBER BY THE INJECTION OF CREOSOTE OIL.

Discussion.*

By Messrs. James C. Haugh, J. L. Campbell, Cliff S. Walker, E. H. Bowser and L. J. Le Conte.

Mr. Haugh. James C. Haugh,† Esq. (by letter).—The writer submits the following observations on creosoted Southern Pine piles, brace plank, stringers, ties and caps, which have been in use on the New Orleans and North-Eastern Railroad since 1883.

The timber and piles were treated according to the specifications in use at that time by the Pascagoula Works, the requisite being 15 lb. per cu. ft. The oil used was principally "London Oil." The piles were driven in brackish water, into which the teredo does not come.

These observations represent the experience gained in 22 years of maintenance, and deal with what appear to be the causes of decay and failure.

Round Piles and Poles.—The depth of the injection in round piles and poles is practically limited to the sap wood. The impregnation of the sap wood of round sticks from which the inner layer of bark is not thoroughly peeled is often irregular and defective. After being subjected to the weather, this inner layer peels off, leaving

* This discussion (of the paper by H. R. Stanford, M. Am. Soc. C. E., printed in *Proceedings* for November, 1905), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Resident Engineer, New Orleans and North-Eastern Railroad.

whitish surfaces. Chips cut from these places are almost void of any oil. Even when a chip is put in one's mouth hardly a taste of the oil is perceptible. Piles or poles should have this layer of unformed sap wood thoroughly removed before treatment. Mr. Haugh.

When the heads of piles are cut to grade, an application of hot creosote oil and also asphalt thinned with oil should be applied to the cut surface. The failures of piles on structures with which the writer is familiar can be attributed to neglect to protect the heads of the piles in this way, and to the use of a 1-in. square bearded drift-bolt. The heart wood decays and leaves the sap wood sound.

Timbers.—Observations on timbers treated for the Lake Pontchartrain Trestle, in 1882 and 1883, show that several varieties of pine timber are still sound and in perfect preservation. No brace plank, whether sap surface or all heart from the center of the log, has decayed, and similar plank laid on the ground for 20 years is sound. In some instances the heart pieces showed but slight penetration. The stringers on this trestle were 6 by 16 in. by 30 ft., and three stringers were packed under each rail. About 6 300 pieces were used, and none has shown any decay, although the quality of the pine varied from the coarsest loblolly, with one sap surface, to the closest and best quality of long-leafed close-grained yellow pine, practically free from sap.

The penetration of oil in these different qualities of pine varied greatly. The loblolly and other coarse-grained stringers show that timber of this quality absorbed a large percentage of oil, and the close-grained yellow pine a much smaller percentage, probably more than 20 lb. per cu. ft. for the coarse-grained, and less than 10 lb. for the close-grained, pine. The caps used were 12 by 14 in. by 14 ft.; the ties, 6 by 8 in. by 9 ft. The guard-rails were 5 by 8 in. and all were of the same varied qualities of pine. Timbers of the same size were treated together.

There were more failures in caps than in timbers of any other size. This, in the writer's opinion, was due to the size of the timbers not admitting of thorough seasoning, consequently, they "checked" afterward. This checking extended beyond the point to which the creosoted oil penetrated.

Almost without exception, the caps had the heart core at or near the center, and the circular grains were not cut across in being sawn, as was the case with stringers, plank and halved and quartered ties.

The stringers were halved from a log in sawing, and all the grains were cut across on the heart face. This permitted more thorough seasoning and the penetration of the oil into the heart face.

The ties were what are known as pole, halved and quartered ties. The pole ties having the heart core in the center also failed by checking beyond the point to which the oil had penetrated.

Mr. Haugh. The halved and quartered ties having the grain cut across admitted of more thorough seasoning and penetration of the oil, and, when placed in the structure with the "sap side" up, there were few cases of decay.

The halved or quartered guard-rails showed the same durability as the ties. The pieces with the heart core in the center failed.

It would seem from these observations that similar timbers, in which the heart core is in the center, cannot be so thoroughly seasoned as to prevent checking and internal decay, even when the creosoted sap surfaces protect it, and that timbers halved and quartered permit of the proper seasoning, and, even where the quantity of oil absorbed is slight, are still sound.

TABLE 2.—REPORT OF TRANSVERSE TEST OF CREOSOTED PINE STRINGER, NO. 1, FROM THE NEW ORLEANS AND NORTH-EASTERN RAILROAD.

Breadth = 6 in. Height = $15\frac{1}{2}$ in. at center. Length between supports = 132 in. Maximum fiber distance = 7.75 in. Moment of inertia = 1 861.9. Load applied in increments of 3 000 lb. Riehlé testing machine used. Time, about 1 hr. 30 min. Date, June 23d, 1905.

Load, in pounds.	Stress per square inch in outer fiber.	Deflection. Reading: R. L.	Deflection. Total, in inches.	Set. Reading: R. L.	Set. Total, in inches.	Remarks.
....	....	0.10 0.13	....	....	....	11 hard rings to the inch.
3 000	....	0.16 0.16	0.045	....	....	
6 000	....	0.20 0.21	0.09	....	....	
9 000	....	0.25 0.26	0.14	....	....	
12 000	....	0.30 0.31	0.19	....	....	
15 000	....	0.36 0.36	0.245	....	....	
18 000	2 472	0.41 0.40	0.29	....	....	Elastic limit.
21 000	....	0.49 0.48	0.37	....	....	For 14 ft. distance between supports, load at elastic limit would be 14 130 lb.
24 000	....	0.56 0.55	0.44	....	....	
27 000	....	0.65 0.62	0.52	....	....	
30 000	....	0.79 0.74	0.65	....	....	
33 000	....	0.94 0.88	0.845	....	....	
						Maximum, 28 500 lb.

	Actual load, in pounds	Deflection, in inches.	Stress per square inch in outer fiber.
Elastic limit....................	18 000	0.29	2 472
Maximum......................	36 300	0.845	4 990

Modulus of elasticity = 1 597 000 lb. per sq. in., at 18 000 lb. load.

Possibly boring a hole longitudinally through the center of timbers containing the heart core would allow better seasoning, prevent to some extent the checking of the surfaces, and admit of the penetration of the oil to the inner rings cut. This boring is now done, at a number of saw-mills, in timbers 20 ft. long. Mr. Haugh.

TABLE 3.—REPORT OF TRANSVERSE TEST OF CREOSOTED PINE STRINGER, NO. 2, FROM THE NEW ORLEANS AND NORTH-EASTERN RAILROAD.

Breadth = 6 in. Height = $15\frac{1}{2}$ in. at center. Length between supports = 132 in. Maximum fiber distance = 7.75 in. Moment of inertia = 1 861.9. Load applied in increments of 3 000 lb. Riehlé testing machine used. Time, about 1 hr. 30 min. Date, June 23d, 1905.

Load, in pounds.	Stress per square inch in outer fiber.	DEFLECTION. Reading: R.	DEFLECTION. Reading: L.	DEFLECTION. Total, in inches.	SET. Reading: R.	SET. Reading: L.	SET. Total, in inches.	Remarks.
....	....	0.10	0.15	....	0.10	0.15	....	6 rings to the inch.
3 000	....	0.14	0.20	0.045	0.10	0.15	0	
6 000	....	0.19	0.24	0.09	0.10	0.15	0	
9 000	....	0.24	0.29	0.14	0.10	0.15	0	
12 000	....	0.28	0.33	0.18	0.10	0.15	0	
15 000	....	0.33	0.38	0.23	0.10	0.15	0	
18 000	....	0.37	0.42	0.27	0.10	0.15	0	
21 000	....	0.42	0.46	0.315	0.10	0.15	0	
24 000	....	0.45	0.53	0.365	0.10	0.15	0	
27 000	3 708	0.51	0.57	0.415	0.10	0.15	0	Elastic limit
30 000	....	0.56	0.64	0.475	0.10	0.17	0.01	For 14 ft. distance between supports, load at elastic limit would be 21 200 lb.
33 000	....	0.63	0.74	0.56	0.12	0.17	0.02	
36 000	....	0.70	0.83	0.64	0.14	0.20	0.045	
39 000	....	0.75	0.90	0.70	......	.....	....	
42 000	....	0.82	1.00	0.785	......	.....	....	
45 000	....	0.90	1.14	0.895	......	......	....	Maximum, 38 900 lb.
48 000	....	1.12	1.45	1.16	......	......	....	

	Actual load, in pounds.	Deflection, in inches.	Stress per square inch in outer fiber.
Elastic limit....................	27 000	0.415	3 708
Maximum......................	49 600	1.16	6 600

Modulus of elasticity = 1 698 000 lb. per sq. in., at 27 000 lb. load.

Where the sizes of timbers required are such that proper seasoning is difficult, if not impracticable, built-up members, which admit of this, should be used.

The cubic contents of the charge to be treated and the measurement of the quantity of oil injected should be ascertained as closely

Mr. Haugh. as practicable, but, in the case of round pine piles, the sap wood varies from, say, 1 in. thick at the butt, in some piles, to 3 in. thick in others, so that the quantity of oil each pile in a load will take varies greatly, and, if thoroughly seasoned and protected, both are equally durable. The same remarks apply to square pine timbers and in these the absorption also varies greatly. The presence of water and the dilution of the fluid should receive close attention.

Mr. Stanford's remarks as to the inspection of certain piles, in July, 1905—five piles being eaten by worms—leads the writer to attribute the failure of the oil to penetrate certain sections to improper "peeling" of the piles.

TABLE 4.—Report of Transverse Test of Creosoted Pine Stringer, No. 3, from the New Orleans and North-Eastern Railroad.

Breadth = 6 in. Height = 16 in. at center. Length between supports = 132 in. Maximum fiber distance = 8 in. Moment of inertia = 2 048. Load applied in increments of 3 000 lb. Riehlé testing machine used. Time, about 1 hr. 30 min. Date, June 24th, 1905.

Load, in pounds.	Stress per square inch in outer fiber.	Deflection.		Set.		Remarks.
		Reading: R. L.	Total, in inches.	Reading: R. L.	Total, in inches.	
....	....	0.05 0.08	....	0.05 0.08	....	5 hard rings to the inch.
3 000	....	0.10 0.13	0.05	0.05 0.08	0	
6 000	....	0.15 0.17	0.095	0.05 0.08	0	
9 000	....	0.21 0.21	0.145	0.05 0.08	0	
12 000	....	0.25 0.26	0.19	0.05 0.08	0	
15 000	....	0.30 0.31	0.24	0.05 0.08	0	
18 000	....	0.36 0.37	0.30	0.05 0.08	0	
21 000	2 700	0.42 0.41	0.35	0.05 0.08	0	Elastic limit.
24 000	....	0.48 0.47	0.41	0.05 0.09	0.005	For 14 ft. distance between supports, load at elastic limit would be 16 500 lb.
27 000	....	0.55 0.52	0.47	0.08 0.10	0.025	
30 000	....	0.61 0.59	0.535	0.08 0.10	0.025	
33 000	....	0.67 0.65	0.595	0.09 0.11	0.035	Maximum, 32 100 lb.
36 000	....	0.75 0.72	0.67	....	....	
39 000	....	0.85 0.82	0.77	....	....	

	Actual load, in pounds.	Deflection, in inches.	Stress per square inch in outer fiber.
Elastic limit....................	21 000	0.35	2 700
Maximum.....................	40 900	0.77	5 260

Modulus of elasticity = 1 405 000 lb. per sq. in., at 21 000 lb. load.

Mr. Haugh.

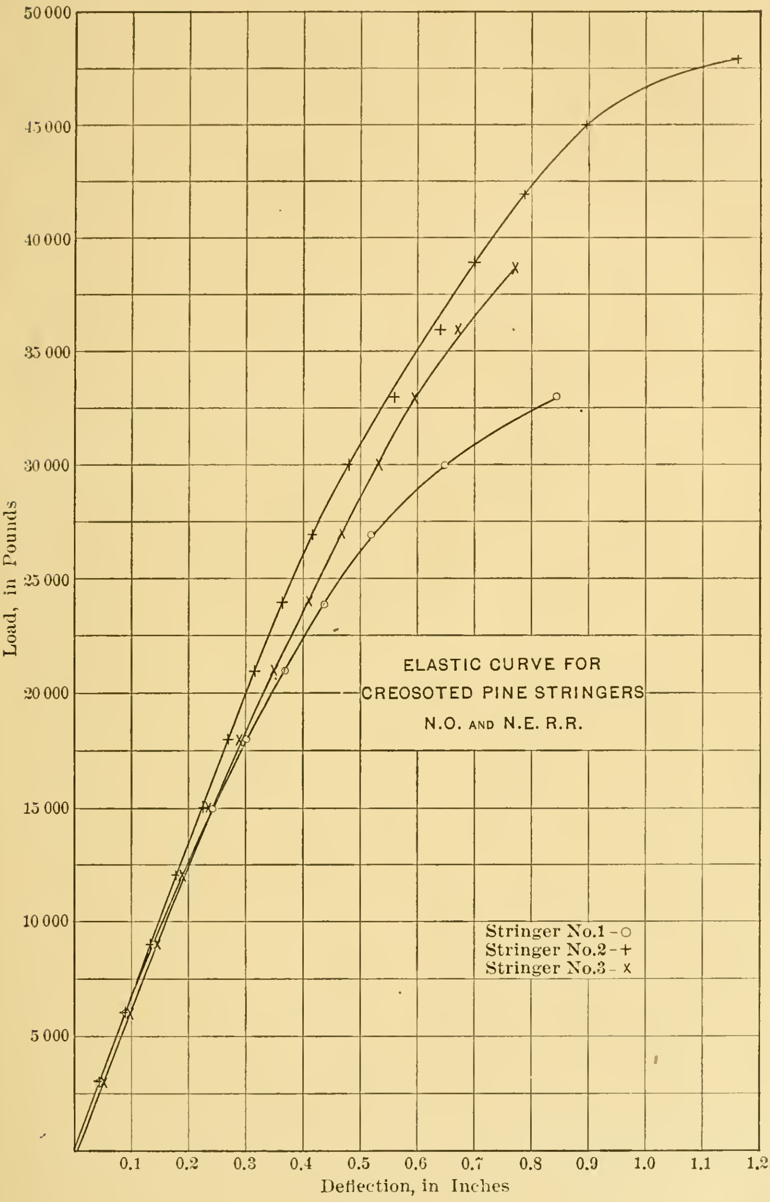

Fig. 1.

Mr. Haugh. The proposed specifications, and the remarks as to air-seasoning, seem to cover the requirements, and the writer would add that if timber is cut from logs which have been floated in creeks or rivers and have been in the water for weeks or months and then air-seasoned, the most desirable seasoning will be obtained. At several creosoting works, this is practicable.

The tests of three pieces of creosoted pine stringers, which were in use on the Lake Trestle from 1883 until taken out in 1905 for test, are given in Tables 2, 3 and 4, and Fig. 1 is a diagram of these tests. The piece, No. 1, was about the average for close-grained yellow pine, having 22 alternate hard and soft grains per inch. The other two pieces are the coarse-grained pine. The penetration of the oil in No. 1 was slight; Nos. 2 and 3 had taken the oil freely. The tests were made by Professor W. B. Gregory, of Tulane University.

Mr. Campbell. J. L. Campbell, M. Am. Soc. C. E. (by letter).—Mr. Stanford's statement that the extent to which creosote oil preserves timber is measured by the quantity of oil and the depth of penetration, while true to the point of saturation with a given grade of oil, should be qualified by the observation that the quality of the oil has quite as much to do with the results as the quantity and penetration.

In the prevailing grade of oil used in the United States the creosote or preserving element constitutes probably not more than 30% of the volume, from which it follows that about 70% of the injection is of no benefit, and that a large total injection is necessary to secure the required quantity of real preservative.

If the oil was refined to a point where the preservative and non-preservative elements were in a proportion inverse to that above given, as obtains in Europe, the life of treated timber would be prolonged very materially. The use of a better grade of creosote opens a most promising field for improvement in the treatment of timber.

The author's statement, that heart wood is practically impervious to oil, is not confirmed by the experience of the El Paso and Southwestern Railroad in its creosoted long-leaf yellow pine bridge timbers, in which the percentage of sap is very small. Stringers having no sap whatever show a required and satisfactory penetration of oil. As to the percentage and penetration of the preservative element in the oil, the writer is not able to say, but there is nothing to indicate that it is all retained near the surface. This timber is treated under contract by creosoting works in Louisiana.

The writer believes the prevailing method of measuring the quantity of oil injected by the tank gauge to be the simplest and best. Leaky pipes and valves and inaccurate gauges are not a valid argument against the method, but rather are favorable to it, as

Mr. Campbell.

pipes and valves can be and should be maintained practically tight to prevent useless waste.

From the author's illustration of the method of measurement by weight, it is quite evident that several very material assumptions have to be made, the possible invalidity of which may explain the inadmissible difference between 22.4 lb. per cu. ft. by tank measure and 13.5 lb. per cu. ft. by estimated weight measure.

Certainly, a system of pipes and valves which will permit the escape and loss of 8.9 lb. of oil out of a total of 22.4 lb. ought to be replaced immediately.

The writer is heartily in favor of air- *vs.* steam- seasoning whenever practicable. He finds that treated ties fail because of disintegration due to brittleness induced by the steam-cooking process of preparation for treatment. Treating plants, however, are generally located in the timber belts, where the normal humidity is great and the air-seasoning process quite slow.

El Paso, Tex., the center of the arid Southwest, with its numerous railway lines (several of them transcontinental), and direct competitive connection with vast, though distant, timber belts, would be an ideal place for the air-seasoning process for timber treatment.

With a grade of oil containing, say, 75% of preservative element, thorough air-seasoning, and proper injection of a sufficient quantity of creosote, sound timber, so treated, should lose very little of its strength and elasticity, and be good for 45 years under ordinary conditions.

Mr. Walker.

CLIFF S. WALKER,* ESQ. (by letter).—The author has given the subject much thought and close investigation, and in the writer's opinion is on the right line to attain the end to be desired by both producer and consumer of creosoted material. Only by thorough injection can timber be preserved, and to approximate that standard of excellence should be the aim of every manufacturer. Poorly treated material has in the past greatly retarded increased demand for preserved timber, and can but injure the future prospects of a growing industry. Any system that tends to give the manufacturer all the profit to which he is entitled, and at the same time protects the consumer, is to be encouraged. The only possible objection to the author's proposed method is that the natural variation in density of timber might cause this method of ascertaining the price to be so risky that in many cases the margin of safety which must be figured would make the cost prohibitive.

To arrive at a fair basis would require long and close scientific investigation, and reports on temperatures, pressures, etc., day and night, should not be left to workmen who lack proper training, or even might be ignorant, lazy or malicious.

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* President, Southern Creosoting Company, Ltd.

Mr. Walker. The writer differs slightly with the author on some points of his proposed specification, principally with the idea of reducing cost of production. In practice, the writer has never found any advantage in treating timber of uniform size and section at one time, and favors applying the higher temperatures, regardless of size of timber, with the intention of shortening the period of treatment. Close observation has convinced him that a better impregnation of timber is secured.

The writer also thinks that the arbitrary specification for oil is unreasonable, as good results have been obtained with various oils differing materially in specific gravity and fractional distillation; and, as the demand for treated timber increases, every possible source of supply for heavy oil of coal-tar should be open.

On the whole, though, the writer is so heartily in favor of the author's views and conclusions, that he would be highly gratified could he find the time to experiment thoroughly on the lines suggested, using the plant of the Southern Creosoting Company in that work, and thus view results obtained, not in the laboratory, but in actual practice, under all conditions and with all classes of material.

Such investigation and experimentation by one so familiar with the work and the conditions to which the finished product is to be subjected would prove of exceeding value.

Mr. Bowser. E. H. Bowser, M. Am. Soc. C. E. (by letter).—This is a much needed paper, in that it touches upon points relating to the subject of creosoting, which, although they have been discussed for many years, are little nearer solution than they were when this method of treating timber was first established.

From their very nature, some of the details of timber treatment can never be brought to any great degree of refinement, though the methods can be improved.

It is well known that sap wood will receive very much more oil than heart wood; that loblolly and old field pine will receive more than long-leaf yellow pine; that in a long pile the small end will receive a greater proportion of oil than the large end, and that the more natural seasoning the material has had, the more rapidly the oil will enter it, and the greater quantity it will hold.

Even after a careful selection of the timber, the quantity of oil per cubic foot injected into each piece will vary greatly, and only a general average treatment can be given, as it is impossible to determine in advance just what material to select for each charge.

By proper inspection, the results of a treatment can be greatly modified and brought much nearer what they should be than with a haphazard method of treating a load without regard to the size, seasoning, or kind of timber.

In considering the inaccurate methods used at present and the

proposed change for determining the quantity of oil injected into timber, all clearly presented in the paper, no very definite results obtained from any long-continued and exhaustive experiments can be given, on account of this part of the subject not having been thoroughly investigated, as yet. Mr. Bowser.

A few experiments which show unexpected results, while they are valuable in that they blaze the lines along which investigation should be made, are not conclusive, and sometimes may even be reversed by further experiments.

The writer will take up some of the causes of the discrepancies in present methods, mentioned in the paper, and a few additional causes, under separate heads.

Calculating the Volume of the Material.—In getting the cubic contents of piles, it is the rule to use the diameters of the large and small ends, measured to the nearest inch. It is not often that the measurement is made to the nearest half inch, and taking intermediate measurements is almost unknown.

The taper of the southern pines is remarkably regular, when proper allowance is made for what is known as "swell butts," and in extra long piles, for the more rapid taper, near the point, on account of the small end having been cut above the branch line.

"Swell butts" are caused by the woodman cutting the tree very close to the ground, in order to get a sufficient diameter, or a longer pile.

To prevent careless measurements of the large end of poles or piles, the Southern Bell Telephone Company, and some others, specify the circumferential measurement 3 ft. from the end.

As the result of a number of measurements taken at different times, and from piles cut in different localities in Mississippi and Louisiana, it has been found by the writer that the taper is very close to an increase of 1 in. in diameter, from the point toward the butt, for every 10 ft. of length.

While the lack of refinement in measuring round timber is often discussed, the errors resulting from taking the mill measurements for unsized sawn timber, in calculating the contents, is usually neglected—in fact, the writer has seen millions of feet of material of this class treated, and not one stick was calculated by the actual dimensions of the cross-section; and, only when the excess in length was over 1 ft., was it usually taken into account; nor would he recommend an exact measurement unless it was stipulated in the specifications, as the bid of the contractor making the treatment is based on the mill size of the section, and the judgment of the inspector is allowed to govern what would be excessive length.

For most purposes for which rough sawn timber is used, the sticks are cut to lengths and the ends squared at the works before treatment, but such is not always the case.

Mr. Bowser. All mills in the Central South set their gauges to saw from $\frac{1}{8}$ to $\frac{3}{8}$ in. full, and this fullness often amounts to $\frac{1}{2}$ in. and even more. This is done to allow for shrinkage after seasoning, irregularities in the alignment of the carriage track, and for the "running" of the saw. Freshly sawn timber will average at least $\frac{1}{4}$ in. full, which in a 12 by 12-in. stick means 4% more than the actual contents. In thin material, such as plank, the variation will average much more. In length, the pieces are seldom cut less than 3 in. full at the mill, and are often more than 6 in. longer than the rated length. Few inspectors will receive a stick which is the least scant in cross-section or length, and, on this account, the variation between the actual and the calculated contents of a charge of sawn timber always makes the cubic content less than it should be.

In the writer's opinion, piles measured at points and butts by the "give and take" method, to the nearest half inch, give much more accurate results than sawn timber taken at the size for which it was cut.

Allowable Quantity of Water in the Oil.—The specifications for oil, quoted in the paper, are, as far as the water is concerned, specifications by which oil should be bought; and it is the common custom to allow $2\frac{1}{2}$% of water, and no more, in the oil from the manufacturer, but, in the writer's judgment, a margin should be allowed for water taken into the oil during the process of treatment at the creosoting works, and at least 5% should be allowed, provided the quantity greater than $2\frac{1}{2}$% is compensated for by an extra injection of oil.

Many specifications allow a maximum of 8%, and this quantity does not seem to be excessive, as the quantity of water would be offset by deeper penetration or by putting the proper quantity of oil in the same space that it would occupy if there were not more than $2\frac{1}{2}$% of water.

With present methods of manipulation, it is not practicable to keep the water to a $2\frac{1}{2}$% limit at all times. Getting water out of oil is expensive, and rigid specifications would, no doubt, cause a corresponding increase in prices if the inspection for water was also rigid.

Waste During Injection, and Loss Due to Reduction of Volume of Oil under Pressure.—Very nearly all the waste oil can be accounted for by collecting the leakage in large pans made for the purpose, and emptying it into the measuring tank at the completion of the treatment. If there is a leak in the steam coils, this is easily discovered, and the set of coils in which it is found can be cut out of service and throttled at the end where the oil would pass out.

The writer has no data at hand showing the elasticity of the oil, and can form no idea as to the reduction of the volume caused by

Mr. Bowser.

the pressure to which it is subjected, but it probably does not amount to very much.

In some specifications the payment for the quantity of oil injected is based on the difference between the readings of the gauge before the oil is turned on the charge and after it is pumped back into the measuring tank. This eliminates any errors due to compression, the oozing out of the oil from the wood after the pressure is released, and very nearly all the loss of oil by waste, if the waste is properly collected.

Such provisions were made in the specifications written by J. F. Coleman, M. Am. Soc. C. E., for the treatment of material for the New Orleans docks.

The Prevailing Method of Measuring the Quantity of Oil Injected.—A number of the creosoting works have measuring tanks 20 ft. in diameter, but tanks proportioned to the sizes of the cylinders would give more uniform results. It has been the writer's experience that a well-constructed sliding gauge, kept in good condition, can be read within less than ¼ in., and a 6-in. treatment from a 20-ft. tank ought not to vary more than 3% either way.

The heavier the treatment, or the larger the cylinder and load, the less will be the variation.

A tank, 30 ft. deep, with a diameter great enough to give a capacity of one and one-quarter times that of the empty cylinder which it supplies, would be about the right proportion, and, with a properly made sliding gauge, ought to give results within 1% of a refined measurement.

This rule would give, for a cylinder 6 ft. in diameter and 100 ft. long, a measuring tank 11 ft. in diameter; and, for a cylinder 9 ft. in diameter and 100 ft. long, a tank 16 ft. in diameter.

The accuracy of the sliding gauge will depend upon the size of the horizontal section of the float, the frictional resistance in the bearings of the pulleys, the pliability of the wire connecting the float with the sliding pointer, and the proper balancing of the pointer so that the guides will not clamp on the gauge-board.

The larger the float the less the distance it will be lifted out of the oil by the friction of the pulleys and the pointer guides.

The pulleys, of course, should be large enough in diameter to prevent the tendency of the wire to form a hook. A light, well-made chain would give better results than a wire.

The wind playing on a long wire running from a measuring tank to a gauge inside the cylinder shed has been observed to make a variation in the reading of the gauge.

The difference between the quantity of oil injected into sawn timber, as indicated by present methods of measurements, may often vary as much as 10% less than the actual amount. This is due to the following causes:

Mr. Bowser. The difference between 8.7 lb. per gal., the weight of the oil at about 75° fahr., which is generally used in calculating the quantity of oil to be injected, and 8.33 lb. per gal., the weight of the oil at 180° fahr., which is about the temperature of the oil in the measuring tank, making a shortage of oil amounting to 4%; the difference in the volume of the oil when in the measuring tank and when under pressure in the cylinder; the running out of some of the oil from the timber after the pressure is released; the fullness of the timber not being taken into account; and the loss by waste, which can be kept very low if proper care is used. There can be considerable loss if care is not taken in analyzing for water.

With proper specifications and proper inspection, all the foregoing discrepancies can be reduced to a very small percentage.

Proposed Method, of Estimating the Quantity of Oil Injected, by Full-Sized Test Pieces.—The most radical departure brought forward in Mr. Stanford's paper is the proposed change in the method of determining the quantity of oil injected.

The quantity of oil which different pieces of timber will absorb varies greatly with the texture, the quantity of resin in the ducts, the quantity of sap wood, the seasoning, the relation between thickness and breadth, and the length.

It is so well known that sap wood will absorb very much more oil than heart wood that nothing further need be said. The variation of the quantity of oil that can be absorbed on account of different degrees of seasoning is very great.

In a 16-lb. treatment, at one works, a load of branch pine piles, 50 ft. long, which had been seasoning for 6 or 8 months, was given a treatment in about one-third the time usually taken for freshly cut timber. The inspector allowed three piles, only a few days from the woods, to be put in with the seasoned piles, and, after treatment it was proved by boring that the latter piles were well treated to the center and the former were penetrated by the oil about 1 in. only, and that that space was not very well saturated. This, of course, is an extreme case.

The writer has often observed that the oil can be injected into piles which have had only a week or two of seasoning much more rapidly than when the piles are put into the cylinder fresh from the stump, or are taken out of water storage.

Piles allowed to lie in the sun, on ground more or less wet, will not only show a different degree of seasoning on the upper and lower sides, but, after creosoting, will show very plainly a difference of saturation. This, no doubt, was the cause of the irregular treatment of the piles eaten by marine worms at the Pensacola Navy Yard. These piles were from West Pascagoula, and nearly all were delivered at the works by water. In hauling them out, for sorting

an order, while most of them were put on skids, some were allowed to lie on the wet ground without occasional turning. Mr. Bowser.

The relation between the cross-section and the cubic contents of timber gives large variation in the absorption. The absorptive power of a 12 by 12-in. stick, compared with a 1-in. plank, when the treatment would give about ½ in. of penetration, would be as 16 to 100.

In sticks of the same cross-section, the shorter ones will absorb more, on account of the penetrating power of the oil being greater in the ends of the fibers. In heavy treatments the oil will penetrate as much as 1 ft. into the end of heart wood if it is not very resinous, and it will sometimes penetrate 5 or 6 ft. into the ends of sticks of loblolly and old field pine.

No matter how carefully a charge is inspected, it is only possible to get an average treatment. The writer doubts that human ability can select from a promiscuous pile of round or square timber a cylinder load from which two or three or all the pieces could be weighed before and after treatment so as to give a basis of measurement for oil which would not often vary as much as 100% or more, from the actual quantity in the timber. Future investigation may show whether or not this doubt is correct.

Here is a case in point, where 80-ft. piles were gauged by 5-ft. test pieces: In the calculations for the quantity of oil to be injected into the piles, 12% additional was allowed for discrepancies in the measuring system. On account of the short pieces having a much greater proportion of end wood exposed than the piles, the result should have shown a greater quantity of oil than by the tank measurement, if the absorbing power of the different piles was approximately equal. The allowance for the inaccuracies of the measuring system would seem to be very close to what it should have been, and, no doubt, very nearly the correct quantity of oil to treat the timber properly passed from the measuring tank into the cylinder, but, according to the test pieces, 37½% of the oil was missing. From the construction of the plant, the oil must have gone either into the wood or into the underground dumping tank through leaking valves, and the only point to be settled is—which?

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The writer's experience in this matter has been confined to operations on the Pacific Coast, where the *modus operandi* is quite different, in many respects; but, nevertheless, so far as he is able to judge, the final results seem to be about the same as for the method reported by the author. Mr. Le Conte.

In 1890, when creosoting works were fairly started in California, the writer was classed among the stalwart advocates of this process of preserving timber. He was untiring in his efforts to reduce the

Mr. Le Conte. uncertainties of the process to a minimum, and, with this end in view, studied carefully the minutest details of operations from the very beginning to the final taking out of the finished product. After the most thorough research he is free to state that the very best creosoting works in the country, using the very best grade of oil and performing the operations in the most thorough and conscientious manner cannot turn out a uniformly good product. That is to say, the general output of the works, figuratively speaking, may be classified as follows, according to the degrees of imperfection:

First.—One-quarter will be found, on examination, to be first-class in every particular, with no defects.

Second.—One-quarter will be slightly imperfect, but would easily pass inspection.

Third.—One-quarter will barely pass inspection.

Fourth.—One-quarter will not pass inspection at all, and would have to be put into the boiler for a second dose, or sold to some one who is not so particular. All four classes have had the same treatment, administered by the same competent men and at the same time, and yet the final results are widely different. There can be but one explanation for this unavoidable state of affairs, and that is, the natural variations in the physical character of the timber; nothing else will account for it. It is extremely difficult to cull out the inferior timber before preservation, and, as a rule, it is never done.

The natural variations can be brought out most graphically by taking a condemned pile, cutting it into 2-ft. lengths, and then critically examining the sections made by the saw. Many years ago, when the writer first looked at them, they threw a hopeless cloud of doubt about the efficiency of the entire process. Long experience, however, has toned down these unavoidable difficulties very materially, so that now if a uniformly good job is demanded the timber must be fresh cut green timber, free from physical defects, and selected with the greatest possible care. Even then, physical defects will crop out in spite of every precaution. In the hurry of every-day business, and especially when the superintendent of the works has a rush order, care in selection of material is simply out of the question; at all events, such care is never taken, disappointment is sure to follow in a few months, and the creosoting plant is given a black eye, so to speak.

In preparing specifications, there is just one thing to keep constantly in mind, and that is, one cannot, by any set of specifications, obtain a better product than the creosoting plant is physically able to produce. This is the business limit beyond which one cannot advance. Therefore, in preparing specifications, the first thing to find out is what the creosoting plant is physically able to do; then

Mr. Le Conte.

the specifications should be framed so as to compel the works to comply with them.

In California the treatment is quite different from that described by the author, and a brief statement may be of interest.

The timber to be preserved is loaded on heavy iron trucks and run into the boiler, and then the ends are closed tight.

All the upper cocks are then closed and the three bottom cocks are opened and the vacuum pumps started. The hot creosote oil (130° fahr.) rises from the ground-tanks and gradually fills the boiler. The foreman watches the filling by feeling the rise of the hot line on the shell of the boiler until it is nearly full of hot oil. Then he stops the vacuum pumps, closes the three bottom cocks and opens a 2-in. safety cock on top of the boiler. He then completes the filling of the boiler with an auxiliary force pump and watches the filling until complete, by means of the safety cock.

When the boiler is full, superheated steam is turned into the steam coils, in the lower half of the boilers, the temperature of the contents being raised from 130° to 220° fahr., and maintained at that temperature for a period of about 10 hours. The vapors of sap and moisture from the timber are blown off through the safety cock on top of the boiler. As long as the sap vapors are rising and discharging the temperature is easily held at 220° fahr., but as soon as these vapors are all driven off, the temperature rises rapidly and the vapors of naphthalene begin to blow off, and, condensing, fall like snowflakes about the boiler room. Vaporization is finished. Auxiliary pumps are started once more and the boiler entirely filled.

All cocks of every description are now closed tight, and, for the first time since operations began, the pressure process begins. The measured quantity of oil, previously calculated, is then forced in with force pumps, the time required depending upon the character of the timber being treated, generally from 4 to 5 hours. The pumps keep up a steady pressure of 150 lb. per sq. in. on the boiler, and the steam coils below maintain a steady temperature of about 200° fahr. When the measured quantity is forced in, the process is completed.

The total time of treatment from beginning to end generally approximates 16 hours. The depth of penetration and quantity of dead oil are the main features. On the Pacific Coast the specifications generally call for 12 to 14 lb. per cu. ft. The writer prefers heavier doses; and, furthermore, that the penetration of the black oil shall not be less than 1 in. in depth. This requirement arises from the fact that the oil, while being forced into the timber by pressure, undergoes a mechanical separation, the lighter and more fluid tar-acids and naphthalene penetrate through the full depth of the sap wood, while the heavier portions, mostly the residuum, remain near

Mr. Le Conte. the surface. It is the latter which, to his mind, constitutes the main protection against the teredo.

The author refers to the danger of the dilution of oil with from 19 to 24% of water. This danger could hardly arise in the California process, in which the green timber is boiled in oil at 220° fahr. for 10 hours at a stretch, or until all watery vapors disappear. This is the highest temperature to which the timber is subjected at any time. The author's suggestion that the weight of creosote due to impregnation is more reliable than the volumetric tank method now in vogue would hardly be practicable in the California practice.

The author's experience at Pensacola, where only 5 piles out of 198 were badly worm-eaten after 15 months' exposure, seems to the writer to be a very fair record, indeed.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the Volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WILLIAM MARSHALL REES, M. Am. Soc. C. E.*

DIED DECEMBER 4TH, 1905.

William Marshall Rees was born at Stroudsburg, Pennsylvania, on December 24th, 1851, and died at Memphis, Tennessee, on December 4th, 1905.

He was graduated from Lehigh University in 1874 at the head of his class. Almost immediately after graduation Mr. Rees went with the East Sugar Loaf Colliery, of Stockton, Pennsylvania, as Assistant Superintendent and Engineer, and remained with them in that capacity until July, 1875, when he left to accept the position of Superintendent of the Humboldt Colliery at Hazleton, Pennsylvania.

In January, 1877, he left Hazleton, to engage in railroad and mining work. During 1877-78 he located and constructed the Stroudsburg and Bethlehem Railroad. During the same time he was Mining Engineer for G. B. Linderman and General Manager of the Bethlehem Iron Company.

In December, 1878, Mr. Rees left his native State to go south and engage in Government work on the Mississippi River under the Mississippi River Commission, being attracted by the high scientific character of the work.

During the first three years of his Government service, he was engaged on surveys, examinations, gauging the river and gathering other data on which to base plans for its improvement. During 1881 he was engaged in constructing snagboats to be operated on Red River. In 1882 he went with the Pratt Coal and Coke Company, of Birmingham, Alabama, as Superintendent. In 1883 he returned to the Government service on the Mississippi River and, as Principal Assistant Engineer, had charge of all channel work in the First and Second Districts. During his Government service he designed and constructed various floating plants used in connection with channel improvement.

In the latter part of 1889 Mr. Rees went with the Sanitary District of Chicago as Assistant Chief Engineer. Upon the resignation of the Chief Engineer, L. E. Cooley, M. Am. Soc. C. E., Mr. Rees also resigned. He then returned to his old position in the Govern-

* Memoir prepared by W. M. Gardner, M. Am. Soc. C. E.

ment service, where he remained until his death, the result of injuries received in the performance of duty.

During his different engagements Mr. Rees did a good deal of expert work, being eagerly sought by those desiring such advice, which he was eminently fitted to give. He was a man of broad education, and possessed a wonderful fund of information in all branches of the profession, which he was ever ready to impart to the younger members.

It has been the writer's good fortune to be associated with Mr. Rees for the past ten years, and he feels himself indebted to him for many kindnesses received from the helping hand so generously extended to smooth over rough places.

Mr. Rees was a Charter Member and Past-President of the Memphis Engineering Society. He was elected a Member of the American Society of Civil Engineers on October 4th, 1905.

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

February, 1906.

PROCEEDINGS = VOL. XXXII—No. 2

HERMAM W SPOONER

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PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL. XXXII. No. 2.

FEBRUARY, 1906.

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1906.

Vol. XXXII. FEBRUARY, 1906. No. 2.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS.

CONTENTS:

MINUTES OF MEETINGS.

OF THE SOCIETY.

FIFTY-THIRD ANNUAL MEETING.*

January 17th, 1906.—The meeting was called to order at 10 A. M.; President C. C. Schneider in the chair, T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, about 350 members.

The reading of the minutes of the meeting of January 3d was dispensed with.

Messrs. W. D. Kelley, George L. Wilson and J. A. Knighton were appointed tellers to canvass the Ballot for Officers for the ensuing year.

* A full report of the Fifty-third Annual Meeting is printed on pages 49 to 84 of this number of *Proceedings*.

The Annual Report of the Board of Direction and the Annual Reports of the Secretary and of the Treasurer,* for the year ending December 31st, 1905, were presented, and, on motion, duly seconded, accepted and placed on file.

A progress report,† from the Special Committee on Uniform Tests of Cement, was presented by Richard L. Humphrey, M. Am. Soc. C. E., Secretary of that Committee.

On motion, duly seconded, the report was received, placed on file, and the Committee continued.

A majority report, and also minority reports,‡ from the Special Committee on Rail Sections, were presented by Robert W. Hunt, M. Am. Soc. C. E., Secretary of that Committee.

On motion, duly seconded, the report was received, placed on file, and the Committee continued.

A progress report,§ from the Special Committee on Concrete and Reinforced Concrete, was presented by Richard L. Humphrey, Secretary of that Committee.

On motion, duly seconded, the report was received, placed on file, and the Committee continued.

The following proposed amendments to the Constitution were then considered:

Amend Article II, Section 2, as follows:

Insert after the word "Civil" in the first line "Engineer who shall have reached a position of recognized standing in the profession, in its several branches including." Also strike out all of the second line beginning with the word "Electrical," and insert "and Electrical Engineering or in Architecture or Marine Architecture." Also strike out all after the word "age" in the fourth line.

The section will then read:

"2. A Member shall be a Civil Engineer who shall have reached a position of recognized standing in the profession, in its several branches, including Military, Naval, Mining, Mechanical and Electrical Engineering, or in Architecture or Marine Architecture. He shall be at the time of admission to membership not less than thirty years of age."

This amendment was proposed by Messrs. James Owen, Ralph H. Chambers, Philip W. Henry, Walter H. Sears, S. Whinery, A. P. Boller and Foster Crowell.

The Assistant Secretary read letters relating to this amendment from Messrs. Charles H. Ledlie, L. S. Randolph, Dugald C. Jackson, Benjamin Thompson, Edward M. Boggs, Ellis B. Noyes, Joseph Lillich, A. B. Wood, Robert L. Lund and Mark L. Ireland.

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* The Annual Reports of the Board of Direction, the Secretary and the Treasurer may be found on pages 7 to 18 of the *Proceedings* for January, 1906 (Vol. XXXII.)

† See page 49.

‡ See page 50.

§ See page 64.

S. Whinery, M. Am. Soc. C. E., presented the following motion:

"That the amendments be referred to a committee of five Corporate Members, to be appointed by the President, which committee shall report to the next Annual Convention what, if any, changes they deem advisable in the present classification of the membership of the Society, or in the qualifications for each grade of membership; and submit any amendments to these amendments which they may recommend."

This motion was seconded by Foster Crowell, M. Am. Soc. C. E.

E. P. North, M. Am. Soc. C. E., moved in amendment that the five Corporate Members mentioned in the motion be increased by one Associate and one Junior, which amendment was accepted.

The motion was then put to vote and carried unanimously.

The following were appointed members of the Nominating Committee to serve two years:

JOHN H. COOK..........	*Representing*	*District*	*No.*	*1.*
LEONARD METCALF.......	"	"	"	*2.*
A. H. SUTERMEISTER.....	"	"	"	*3.*
EDGAR MARBURG.........	"	"	"	*4.*
JOHN W. ALVORD........	"	"	"	*5.*
A. H. ZELLER...........	"	"	"	*6.*
FRANK O. MARVIN.......	"	"	"	*7.*

The Assistant Secretary reported that the Board of Direction had awarded the prizes for the year ending with the month of July, 1905, in accordance with the recommendations of the Committee appointed for that purpose, as follows:

That the Norman Medal be awarded to Paper No. 997, "The Structural Design of Buildings," by C. C. Schneider, President, Am. Soc. C. E.

That the Thomas Fitch Rowland Prize be awarded to Paper No. 981, "Lake Cheesman Dam and Reservoir," by Charles L. Harrison, M. Am. Soc. C. E., and Silas H. Woodard, Assoc. M. Am. Soc. C. E. (now M. Am. Soc. C. E.).

That the Collingwood Prize be awarded to Paper No. 983, "Lateral Earth Pressures and Related Phenomena," by E. P. Goodrich, Jun. Am. Soc. C. E. (now M. Am. Soc. C. E.).

The Assistant Secretary announced that the Thirty-eighth Annual Convention would be held at The Hotel Frontenac, Thousand Islands, New York, on June 26th to 29th, 1906.

The Assistant Secretary announced the death of Francis Edward Snyder, elected Member, September 6th, 1905; died December 23d, 1905.

The report of the tellers* appointed to canvass the Ballots for Officers for the ensuing year was presented.

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* See page 73.

The President announced the election of the following officers:

President, to serve one year:

Frederic P. Stearns, Boston, Mass.

Vice-Presidents, to serve two years:

Onward Bates, Chicago, Ill.
Bernard R. Green, Washington, D. C.

Treasurer, to serve one year:

Joseph M. Knap, New York City.

Directors, to serve three years:

George Gibbs, New York City.
J. Waldo Smith, New York City.
Emil Swensson, Pittsburg, Pa.
James M. Johnson, Louisville, Ky.
Wynkoop Kiersted, Kansas City, Mo.
William B. Storey, Jr., Topeka, Kans.

Mr. Green and Mr. Swensson conducted Mr. Stearns, the President-elect, to the chair.

Mr. Stearns addressed the meeting briefly.

Adjourned.

February 7th, 1906.—The meeting was called to order at 8.35 p. m.; President Frederic P. Stearns in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 109 members and 26 guests.

The minutes of the meeting of January 3d, 1906, were approved as printed in the *Proceedings* for January, 1906.

A paper, entitled "Test of a Three-Stage, Direct-Connected Centrifugal Pumping Unit," by Philip E. Harroun, M. Am. Soc. C. E., was presented by the Assistant Secretary, who also read a written communication on the subject from Elmo G. Harris, M. Am. Soc. C. E. The paper was discussed further by H. F. Dunham, M. Am. Soc. C. E.

Ballots for membership were canvassed, and the following candidates elected:

As Members.

George Gray Anderson, Denver, Colo.
Walter Francis Ballinger, Philadelphia, Pa.
Sverre Dahm, New York City.

George Tillinghast Hammond, New York City.
John Farnsworth Hammond, New York City.
William Huggins, Sao Paulo, Brazil.
Andrew Murray Hunt, San Francisco, Cal.
William Henry Hunter, Manchester, England.
Luigi Luiggi, Rome, Italy.
William Edwin Moore, Clarkston, Wash.
William Mueser, New York City.
Henry Irwin Randall, Berkeley, Cal.

As Associate Members.

James Ray Aikenhead, East Liverpool, Ohio.
De Witt Dukes Barlow, Cape May City, N. J.
Lorenzo Dana Cornish, Beaver, Pa.
George Washington Craig, Omaha, Nebr.
Frederick William Dencer, Chicago, Ill.
Franklin Edward Estes, Rincon Antonio, Oaxaca, Mexico.
Maurice Goldenberg, Detroit, Mich.
Edmund Ryond Halsey, Newark, N. J.
Harry Garfield Harrington, New York City.
George Rogers Heckle, Columbus, Ohio.
Henry Detrick Jouett, New York City.
John Edward Kirkham, Ambridge, Pa.
Egbert Jessup Moore, New York City.
Arthur Tappan North, Chicago, Ill.
Chikao Oinouye, Steelton, Pa.
Albert Henry Perkins, Chinook, Mont.
John Richards Pill, Carbon Hill, Ala.
Paul August Schuchart, New York City.
Willard Wilberforce Stone, Syracuse, N. Y.
Frank Hamant Trow, Clinton, Mass.
Earle Hubbel Welles, Cleveland, Ohio.
Ezra Bailey Whitman, New York City.
Samuel Walter Williams, New York City.
Charles Sumner Williamson, Pittsburg, Pa.

As Associates.

Coleman Meriwether, New York City.
Albert Moyer, New York City.
John McGaw Woodbury, New York City.

The Assistant Secretary announced the transfer of the following candidates, by the Board of Direction, on February 6th, 1906:

From Associate Member to Member.

Ricardo Manuel Arango, Panama, Panama.
George Thomas Barnsley, Pittsburg, Pa.
Edward Francis Haas, San Francisco, Cal.
Eugene Lentilhon, New York City.
Fred William Lepper, Cleveland, Ohio.
Amos Schaeffer, New York City.

The election of the following candidates, by the Board of Direction:

As Juniors.

On September 5th, 1905:

Frank C. Huntsman, Macon, Mo.

On December 5th, 1905:

Leon Lincoln Gay, Minidoka, Idaho.

On January 2d, 1906:

Francis Winfield Collins, New York City.
Charles Houchin Higgins, Jersey City, N. J.
Harold Scott Loughran, New Rochelle, N. Y.
Alfred Marshall Wyman, East Orange, N. J.

On February 6th, 1906:

Lewis Paul Bremer, New Rochelle, N. Y.
Elmer George Brua, Monterey, Cal.
Francis Stirling Crowell, Flushing, N. Y.
William Earle Elam, Empire, Canal Zone, Panama.
Fritz Louis Metzger, Freedom, Pa.
Kenneth Dunham Owen, Montclair, N. J.
Lionel Henry Peabody, Jr., Providence, R. I.
Lafayette Clowe Reynolds, Mt. Vernon, N. Y.
Albert Irvine Stiles, Lima, Peru.
Moritz Wormser, New York City.

The Assistant Secretary announced the following deaths:

Carl Christian Adolph Both, elected Member, September 2d, 1891; died January 12th, 1906.

Joseph Hockman Bowman, elected Associate Member, December 2d, 1903; date of death unknown.

Adjourned.

February 14th, 1906. Extra Meeting.—The meeting was called to order at 8.40 P. M.; Vice-President Kuichling in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 65 members and 20 guests.

A lecture, entitled "Telephone Line Engineering," was presented by C. J. H. Woodbury, M. Am. Soc. C. E., and illustrated with lantern slides.

A vote of thanks to Mr. Woodbury for his interesting lecture was passed unanimously.

February 21st, 1906.—The meeting was called to order at 8.40 P. M., Charles S. Gowen, Director, in the chair; F. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 103 members and 31 guests.

A paper by John S. Sewell, M. Am. Soc. C. E., entitled "The Economical Design of Reinforced Concrete Floor Systems for Fire-Resisting Structures," was presented by the author.

The paper was discussed orally by Messrs. E. P. Goodrich, J. Kahn and H. T. Forchhammer, and illustrated with lantern slides.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

January 2d, 1906.—President Schneider in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bissell, Bowman, Craven, Croes, Curtis, Gowen, Knap, N. P. Lewis, and Osgood.

Action was taken in regard to members in arrears for dues.

The following report was received from the Committee on Award of Prizes:

"PHILADELPHIA, PA., DEC. 26TH, 1905.

"To the Board of Direction,
AMERICAN SOCIETY, CIVIL ENGINEERS,
NEW YORK.

"GENTLEMEN:—Your Committee appointed for the purpose, recommend the award of prizes for papers published during the year ending July 1st, 1905, as follows:

"The Norman Medal to Mr. C. C. Schneider, for Paper No. 997, 'The Structural Design of Buildings.'

"The Thomas Fitch Rowland Prize to Messrs. Charles L. Harrison and Silas H. Woodard, for Paper No. 981, 'Lake Cheesman Dam and Reservoir.'

"The Collingwood Prize for Juniors to Mr. E. P. Goodrich, for Paper No. 983, 'Lateral Earth Pressures and Related Phenomena.'

"JAMES CHRISTIE,
"J. T. FANNING,
"F. C. KUNZ,
"Committee."

The Norman Medal, The Thomas Fitch Rowland Prize, and the Collingwood Prize for Juniors were awarded in accordance with the recommendations of the foregoing report.

The following resignations were accepted as taking effect December 31st, 1905:

MEMBERS:
ERNESTO J. BALBIN,
ALFRED WILLARD FRENCH.

ASSOCIATE MEMBERS:
GEORGE H. CRAFTS,
RALPH PEVERLEY,
EDMUND P. RAMSEY.

Applications were considered and other routine business transacted.

Two Associate Members were transferred to the grade of Member, and thirteen candidates for Junior were elected.

Adjourned.

January 17th, 1906.—The Board met during the Annual Meeting, as required by the Constitution, President Frederic P. Stearns in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, Messrs. Bissell, Bowman, Ellis, Fisher, Gowen, Green, Lewis, Noble, Schneider, Sherrerd, Smith, and Swensson.

The following Standing Committees were appointed:

Finance Committee: Emil Kuichling, Charles S. Gowen, George Gibbs, M. L. Holman, George S. Pierson.

Publication Committee: Morris R. Sherrerd, J. Waldo Smith, Onward Bates, Bernard R. Green, George S. Webster.

Library Committee: Nelson P. Lewis, A. L. Bowman, Ralph Modjeski, H. Bissell, Chas. Warren Hunt.

A Committee on Membership was also appointed.

A letter-ballot was ordered for the election of a Secretary for the ensuing year.

Frederic P. Stearns, President, Am. Soc. C. E., was selected from the membership of the Society as a Member of the John Fritz Medal Board of Award.

Adjourned.

February 6th, 1906.—8.40 P. M.—President Stearns in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, Messrs. Bissell, Bowman, Ellis, Gibbs, Gowen, Knap, Kuichling, Lewis, Noble, Schneider, and Smith.

Ballots for the election of a Secretary were canvassed, and Chas. Warren Hunt, having received 29 votes, was declared elected.

In compliance with the action of the Annual Meeting, the following Committee on Amendments to the Constitution was appointed by the President: Messrs. Samuel Whinery, Alfred Noble, Onward Bates, Morris R. Sherrerd, and Samuel E. Tinkham, representing the Corporate Members; John C. Trautwine, Jr., representing the Associates, and Thaddeus Merriman, representing the Juniors.

The following resolutions were adopted unanimously:

"*Resolved,* That the thanks of the American Society of Civil Engineers be extended to the Commandant and Officers attached to the New York Navy Yard for their kindness and courtesy in receiving the members of this Society at the Yard, and permitting the inspection of the various buildings, dry docks, etc., the U. S. Battleship *Connecticut,* and other interesting and instructive features."

"*Resolved,* That the thanks of the American Society of Civil Engineers be extended to Gen. Howard Carroll and to the Hon. John H. Starin for their great kindness and courtesy in placing at

the disposal of the Society the steamer *Valley Girl* on the occasion of the Annual Meeting, January 18th, 1906, which enabled a large party of members of the Society to make a very pleasant excursion to the New York Navy Yard and other points of interest."

Applications were considered and other routine business transacted.

Six Associate Members were transferred to the grade of Member, and ten candidates were elected Juniors.

Adjourned.

REPORT IN FULL OF THE FIFTY-THIRD ANNUAL MEETING, JANUARY 17th AND 18th, 1906.

Wednesday, January 17th, 1906.—The meeting was called to order at 10 A. M.; President C. C. Schneider in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, about 350 members. Meeting called to Order.

The President.—The meeting will please come to order. The minutes of January 3d, 1906, in accordance with the custom, will be printed in the January number of *Proceedings*, and come up in due course for action at the meeting of February 7th, 1906. In view of this fact, the reading of the minutes will be dispensed with, unless some call is made for them.

The Chair appoints the following gentlemen as tellers to canvass the Ballots for Officers to be elected at this meeting: Messrs. W. D. Kelley, George L. Wilson and J. A. Knighton. The ballot does not close until twelve o'clock, noon, but to enable a report to be made as soon as possible after that time, the tellers will please proceed with their duty at once. Ballots will be received until twelve o'clock. At that hour the ballot will be declared closed. Tellers Appointed.

The next order of business is the report of the Board of Direction. Report of the Board of Direction.

The Assistant Secretary read the report of the Board of Direction.*

The President.—The report of the Secretary.†

The Assistant Secretary read the report of the Secretary.

The President.—The report of the Treasurer will be read by Mr. Knap.

The Treasurer read his report.‡

The President.—Gentlemen, you have heard the reports of the Board of Direction, the Secretary, and the Treasurer, and if there is no objection to them they will be received and placed on file.

On motion, duly seconded, the reports of the Board of Direction, the Secretary, and the Treasurer were received and placed on file.

The President.—The report of the Special Committee on Uniform Tests of Cement, by Mr. George S. Webster, Chairman. Report of the Cement Committee.

Richard L. Humphrey, M. Am. Soc. C. E.—Mr. Webster, the Chairman of the Committee is unable to be present to-day, and I, as Secretary, have been asked to read the report, which is very brief.

January 17th, 1906.

To the President and Members,

American Society of Civil Engineers.

Gentlemen:

The investigations in progress have not advanced sufficiently to reach definite conclusions, and your Special Committee on Uniform

* See *Proceedings*, Vol. XXXII, p. 7 (January, 1906).
† See *Proceedings*, Vol. XXXII, p. 16 (January, 1906).
‡ See *Proceedings*, Vol. XXXII, p. 15 (January, 1906).

Report of the Cement Committee (continued).

Tests of Cement is unable to present a final report at this time, and it asks, therefore, that it be continued.

G. S. WEBSTER,
Chairman;
RICHARD L. HUMPHREY,
Secretary.

Committee:
GEORGE S. WEBSTER,
RICHARD L. HUMPHREY,
GEORGE F. SWAIN,
ALFRED NOBLE,
LOUIS C. SABIN,
SPENCER B. NEWBERRY,
CLIFFORD RICHARDSON,
F. H. LEWIS,
W. B. W. HOWE.

THE PRESIDENT.—Gentlemen, you have heard the report of the Special Committee on Uniform Tests of Cement. What action do you want taken in regard to it?

It was moved and seconded that the report be received and the Committee be continued.

The motion was carried.

Report of the Committee on Rail Sections.

THE PRESIDENT.—The report of the Special Committee on Rail Sections.

ROBERT W. HUNT, M. Am. Soc. C. E., Secretary of the Special Committee on Rail Sections.—May I request that the Secretary read that report? I am not sure whether or not the Chairman is here.

The report of the Special Committee on Rail Sections was read by the Assistant Secretary, as follows:

REPORT OF THE SPECIAL COMMITTEE ON RAIL SECTIONS.

To the American Society of Civil Engineers,

GENTLEMEN:

Your Special Committee on Rail Sections respectfully report that the instructions under which they were appointed in 1902 are:

1.—To report upon the results obtained in the use of rails of the sections presented to the Society in Annual Convention, August 2d, 1893, by a special committee appointed for that purpose;

2.—To report whether any modification of any of said sections is advisable, and, if so, to recommend such modification;

3.—To report upon the recognized practice as to chemical composition and mechanical treatment used in the manufacture of rails, and the manner of inspection of the same;

4.—To report upon the advisability of the establishment of a

form of specification covering the manufacture and inspection of rails;

5.—If found advisable, to recommend a form of specification for the manufacture and inspection of rails.

Since their appointment, they have held as many meetings as the progress of their work seemed to require; in addition to which, and, in fact, in pursuance of the action of the Committee at those meetings, extensive correspondence has been carried on by the officers of the Committee with the officials of the principal railroads of the United States, Canada, and Mexico. There has also been correspondence with the officers of the Rail Committee of the American Railway Engineering and Maintenance of Way Association, and one joint meeting held with that Committee. There have also been two meetings with a Committee representing the Steel Rail Manufacturers of the United States.

Your Committee would now report:

1.—To report upon the results obtained in the use of rails of the sections presented to the Society in Annual Convention, August 2d, 1893, by a special committee appointed for that purpose:

During 1905 reports have been received from 79 of the leading railroads of the United States, Canada and Mexico.

13 roads had no criticism to make for or against the American Society Sections.

23 roads say they are entirely satisfactory.

2 say emphatically they are good standards.

14 roads criticized the shape of head.

2 roads criticized the web.

2 roads say it ought to have a broader base.

Some other criticisms refer to the vertical sides, which, they claim, wear the wheel flanges, and some other slight changes in shape are suggested.

48 railroads reported that the Society's sections were their standard.

10 railroads reported that they did not use them.

19 railroads have used them partially as standard.

3 railroads have used them, but do not now.

63 railroads say positively they intend to use them as standard.

14 say they will not do so.

2 say they will probably use them.

2.—To report whether any modification of any of said sections is advisable, and, if so, to recommend such modification:

What are popularly known as the rail sections of this Society were recommended by a special Committee in 1893. For the year ending June 30th, 1905, the following percentage of their total out-

Report of the Committee on Rail Sections (continued).

put was rolled by eight mills in the United States for domestic and export uses:

	Domestic.	Foreign.
Pennsylvania Steel Co..... Maryland Steel Co.........	82%	Practically none.
Cambria Steel Co..........	77%	None.
Illinois Steel Co...........	84%	None.
Carnegie Steel Co.........	69.34%	78.9%
Lackawanna Steel Co......	99.3%	Practically none.
Tennessee Coal & Iron Co..	88.6%	None.
Colorado Fuel & Iron Co...	65%	None.

Since 1893 the speed of trains, and wheel loads, have been increased proportionately very much more than the weight of rails. The increase in driving-wheel loads is approximately 60%, while the maximum weight of rails has increased from 80 to 100 lb., or 25 per cent.

Rails of the heavier sections are not giving the service expected of them, even after making due allowance for the increased traffic tonnage or for the trouble caused by badly balanced driving wheels, and poor condition of rolling stock and roadbed, but, after due consideration of all the information collected, your Committee does not feel justified in now recommending any modifications of the sections.

3.—To report upon the recognized practice as to chemical composition and mechanical treatment used in the manufacture of rails, and the manner of inspection of the same:

In relation to the chemical composition of rails made by the acid Bessemer process, the general practice in the United States is to accept what are known as the Manufacturers' Standard Specifications. These are:

	70 lb. up to 80 lb.	80 lb. up to 90 lb.	90 lb. up to 100 lb.
Carbon	0.45 to 0.55	0.48 to 0.58	0.50 to 0.60
Phosphorus, not over.	0.10	0.10	0.10
Silicon, not over.....	0.20	0.20	0.20
Manganese	0.75 to 1.00	0.80 to 1.10	0.80 to 1.10

Previous to December 10th, 1904, the carbon percentages were five points lower, the other elements being amounts as now. Some railroads have insisted and obtained modifications by which the carbon percentages have been increased and the phosphorus kept not to exceed 0.085 per cent. In a few instances, phosphorus has been held lower, but such rails are now made only from a large admixture of imported ores. Some of the Canadian roads have been able to obtain, from the United States rail makers, 80-lb. American Society section rails with the following actual composition:

Carbon	0.58 to 0.64, average 0.60
Phosphorus	0.059 to 0.071
Silicon	0.125 to 0.165
Sulphur	0.049 to 0.056
Manganese	0.93 to 0.96

The Bessemer rails, which the same parties are having manufactured in Canada, contain:

Carbon	0.53 to 0.63, average 0.58
Phosphorus, not to exceed	0.085
Silicon	0.075 to 0.15
Sulphur, not to exceed..	0.075
Manganese	0.80 to 1.10

Up to the present time, it has not been proven that the Basic Bessemer process of rail making is commercially practical with American iron ores. The Basic Open-Hearth process has reached enormous development in the United States, but only one plant is regularly putting its steel into rails. Their standard specifications are:

	70 lb. up to 80 lb.	80 lb. up to 90 lb.	90 lb. up to 100 lb.
Carbon	0.50 to 0.60	0.55 to 0.65	0.58 to 0.68
Phosphorus, not over.	0.06	0.06	0.06
Silicon, not over.....	0.20	0.20	0.20
Manganese	0.75 to 1.05	0.80 to 1.10	0.80 to 1.10

There is also a Basic Open-Hearth rail mill in Canada.

The mechanical treatment of the metal differs somewhat in the practice of the largest American rail producers. At three mills the bloom is reheated after leaving the blooming rolls, and before entering the rail rolls. At the others, the rolling process is a continuous one, from the time the ingots are drawn from the heating furnaces. All but two of the mills have three-high trains of rail rolls. During several years, all the mills have been crowded with work, and the tendency has been to use every exertion to increase production. In your Committee's opinion, this has led to rolling the steel at too rapid a reduction and at too high a temperature, and to other details which largely account for the unsatisfactory service given by the heavy-sectioned rails.

Practically all large purchasers of rails have them inspected at the mills either by men detailed from their own organization or by professional inspectors. Such inspection covers seeing that the provisions of the specifications under which the rails have been purchased are observed; particularly checking the accuracy of section, squareness and length of sawing, accuracy of drilling, straightness in line and surface, and freedom from mechanical defects.

Report of the Committee on Rail Sections (continued).

4.—To report upon the advisability of the establishment of a form of specification covering the manufacture and inspection of rails:

In this country rails are made by two processes—Acid Bessemer and Basic Open Hearth—the former process covering quite 90% of the production. On the continent of Europe nearly all rails are made by the Basic Bessemer, and in England, with the exception of one works, all commercial rails are made by the Acid Bessemer process, the difference in the practice of the several countries being occasioned by the character of their available iron ores. The same controlling influence must be recognized in preparing chemical and other specifications for the manufacture of rails in this country. Your Committee believe that a low percentage of phosphorus and high percentage of carbon make a better rail steel than when the amount of the former element necessitates the curtailment of the latter. When rails of lighter sections were used, chemical conditions were not so important as with the heavier ones; this, because the rolling down of the steel to the smaller sections of necessity gave it more work, and also finished the rail at a lower temperature; thus producing a finer-grained and tougher metal. This partially explains why so many of the early steel rails gave good physical results, while their chemical composition was so irregular.

Taking everything into consideration, your Committee think it advisable to present specifications covering the manufacture and inspection of rails, but realize that restraining commercial ore conditions keep them from being ideal ones, chemically; and existing manufacturing plants and practice limit what can be specified for the physical treatment of the steel; but, believing, as your Committee do, that the physical treatment is of as great importance as the chemical composition, they do recommend certain requirements which should be enforced. These will tend toward the production of better wearing rails, and also safer ones. Your Committee recognize that commercial conditions cannot be entirely disregarded by engineers, but, at the same time, they believe that when it is known that existing practice results in danger to human life and limb, it becomes the duty of the engineer to insist on their being changed, even though that necessitates either a greater cost to the consumer or a somewhat less profit to the producer. That point will adjust itself, but the safer practice should be demanded. This view controlled your Committee in framing the clause of these recommended specifications governing the shearing of the rail blooms, as it is well known that one of the frequent causes of failure of steel rails is due to piping, and that this comes from unsound ingots. Unfortunately, such failures often cause accidents which result in large material damage, and, what is worse, the loss of life. Frequently,

such interior defects cannot be detected until after the rails have been subjected to traffic, hence it is of the greatest importance that care should be exercised in the manufacture with a view of reducing the danger to a minimum.

Your Committee have studied the results obtained from Basic Open-Hearth steel rails, and, while their use has not extended over many years, at the same time, the evidence points to their giving better service than the Bessemer rails. This is strikingly demonstrated by certain experimental very high carbon rails laid on the lines of the Pennsylvania Railroad. It must be understood that the physical differences of the two steels are not entirely due to their chemical composition; as it is a well-known metallurgical fact that steel made by the Basic Open-Hearth process possesses characteristics of its own.

They have prepared specifications for both Bessemer and Basic Open-Hearth rails. While the majority of the rail plants of the country are not now adapted to the making of Basic Open-Hearth rails, the greater known amount of the iron ore supply is suitable, hence the production of that kind of steel rails will increase.

5.—If found advisable, to recommend a form of specification for the manufacture and inspection of rails:

Recommended Specifications for Bessemer Steel Rails.

Process of Manufacture.—The entire process of manufacture and testing shall be in accordance with the best state of the art, and the following instructions shall be faithfully executed:

Ingots shall be kept in a vertical position in the pit heating furnaces until ready to be rolled, or until the metal in the interior has had time to solidify.

No bled ingots shall be used.

There shall be sheared from the end of the blooms formed from the top of the ingots, assuming that such blooms are about 8 by 8 in. square, at least 40 in., and if, from any cause, the steel does not then appear to be solid, the shearing shall continue until it does. If, by the use of any improvements in the process of making ingots, the defect known as piping shall be prevented, the above shearing requirements may be modified.

The number of passes and speed of train shall be so regulated that on leaving the rolls at the final pass, the temperature of the rail will not exceed that which requires a shrinkage allowance at the hot saws, for a 33-ft. rail of 100-lb. section, of $6\frac{7}{16}$ in., and $\frac{1}{16}$ in. less for each 5-lb. decrease of section. These allowances to be decreased at the rate of $\frac{1}{90}$ in. for each second of time elapsed between the rail leaving the finishing rolls and being sawn. No artificial means of

Report of the Committee on Rail Sections (continued).

cooling the steel shall be used after the rails leave the rolls, nor shall they be held before sawing for the purpose of reducing their temperature.

Chemical Composition.—Rails of the various weights per yard specified below shall conform to the following limits in chemical composition:

	70 to 79 lb. Percentage.	80 to 89 lb. Percentage.	90 to 100 lb. Percentage.
Carbon	0.50 to 0.60	0.53 to 0.63	0.55 to 0.65
Phosphorus shall not exceed	0.085	0.085	0.085
Silicon " " "	0.20	0.20	0.20
Sulphur " " "	0.075	0.075	0.075
Manganese	0.75 to 1.00	0.80 to 1.05	0.80 to 1.05

Drop Test.—One drop test shall be made on a piece of rail, not less than 4 ft. and not more than 6 ft. long, selected from each blow of steel. The test piece shall be taken from the top of the ingot. The rails shall be placed head upward on the supports, and the various sections shall be subjected to the following impact tests under a free falling weight:

70 to 79-lb. rails......................	18 ft.
80 to 89-lb. rails......................	20 "
90 to 100-lb. rails......................	22 "

If any rail breaks, when subjected to the drop test, two additional tests may be made of other rails from the same blow of steel, also taken from the top of the ingots, and if either of these latter rails fail, all the rails of the blow which they represent will be rejected, but if both of these additional test pieces meet the requirements, all the rails of the blow which they represent will be accepted.

The drop-testing machine shall have a tup of 2 000 lb. weight, the striking face of which shall have a radius of not more than 5 in., and the test rail shall be placed head upward on solid supports 3 ft. apart. The anvil block shall weigh at least 20 000 lb., and the supports shall be part of, or firmly secured to, the anvil. The report of the drop test shall state the atmospheric temperature at the time the test was made.

Section.—Unless otherwise specified, the section of rail shall be the American Standard, recommended by the American Society of Civil Engineers, and shall conform, as accurately as possible, to the templet furnished by the railroad company, consistent with the paragraph relative to specified weight. A variation in height of $\frac{1}{64}$ in. less, or $\frac{1}{32}$ in. greater than the specified height, and $\frac{1}{16}$ in. in width will be permitted. The section of rail shall conform to the finishing dimensions.

Weight.—The weight of the rails will be maintained as nearly as possible, after complying with the preceding paragraph, to that specified in contract. A variation of one-half of 1% for an entire order will be allowed. Rails will be accepted and paid for according to actual weights.

Length.—The standard length of rails shall be 33 ft. Ten per cent. of the entire order will be accepted in shorter lengths, varying by even feet to 27 ft., and all No. 1 rails less than 33 ft. long shall be painted green on the ends. A variation of ¼ in. in length from that specified will be allowed.

Drilling.—Circular holes for splice-bars shall be drilled in accordance with the specifications of the purchaser. The holes shall conform accurately to the drawing and dimensions furnished, in every respect, and must be free from burrs.

Straightening.—Care must be taken in hot-straightening the rails, and it must result in their being left in such a condition that they shall not vary throughout their entire length more than 5 in. from a straight line in any direction, when delivered to the cold-straightening presses. Those which vary beyond that amount, or have short kinks, shall be classed as second-quality rails and be so stamped.

Rails shall be straight in line and surface when finished—the straightening being done while cold—smooth on head, sawed square at ends, variation to be not more than $\frac{1}{32}$ in., and, prior to shipment, shall have the burr occasioned by the saw cutting removed, and the ends made clean. No. 1 rails shall be free from injurious defects and flaws of all kinds.

No. 2 rails shall be accepted up to 5% of the whole order. They shall not have flaws in their heads of more than ¼ in., or in the flange of more than ½ in. in depth, and, in the judgment of the inspector, these shall not be so numerous or of such a character as to render them unfit for recognized second-quality rail uses. The ends of No. 2 rails shall be painted white, and shall have two prick-punch marks on the side of the web near the heat number brand, and placed so as not to be covered by the splice-bars. Rails from heats which failed under the drop test shall not be accepted as No. 2 rails.

Branding.—The name of the maker, the weight of the rail, and the month and year of manufacture shall be rolled in raised letters on the side of the web; and the number of the blow shall be plainly stamped on each rail where it will not subsequently be covered by the splice-bars.

Inspection.—The inspector representing the purchaser shall have free entry to the works of the manufacturer at all times when the contract is being filled, and shall have all reasonable facilities afforded him by the manufacturer to satisfy him that the finished

Report of the Committee on Rail Sections (continued).

material is furnished in accordance with the terms of these specifications. All tests and inspection shall be made at the place of manufacture prior to shipment.

The manufacturer shall furnish the inspector, daily, with carbon determinations for each blow, and a complete chemical analysis every 24 hours, representing the average of the other elements contained in the steel, for each day and night turn. These analyses shall be made on drillings taken from small test ingots.

For Basic Open-Hearth Rails.—The specifications for rails made by the Basic Open-Hearth process shall be the same as for Bessemer rails, excepting that their chemical composition shall be:

	70 to 79 lb Percentage.	80 to 89 lb. Percentage.	90 to 100 lb. Percentage.
Carbon	0.53 to 0.63	0.58 to 0.68	0.65 to 0.75
Phosphorus shall not exceed	0.05	0.05	0.05
Silicon " " "	.0.20	0.20	0.20
Sulphur " " "	.0.06	0.06	0.06
Manganese	0.75 to 1.00	0.80 to 1.05	0.80 to 1.05

We respectfully submit the above report and request the discharge of your Committee.

Joseph T. Richards, *Chairman;*
C. W. Buchholz,
E. C. Carter,
S. M. Felton,
Richard Montfort,
H. G. Prout,
Edmund K. Turner,
Robert W. Hunt, *Secretary.*

The undersigned concurs in the foregoing report with the exception of the clause relating to the continuance of the present sections.

John D. Isaacs.

Mr. George E. Thackray, upon consideration following the meeting at which the majority report was prepared, decided that he did not feel justified in signing that report, neither did he feel like making a minority report, and, therefore, thought it wise that the report be withheld for further consideration.*

Robert W. Hunt,
Secretary, Special Rail Committee.

* Subsequent to the presentation of the report of the Special Committee on Rail Sections at the Annual Meeting, Mr. George E. Thackray submitted a minority report, and, in order that the views of all members of that Committee be presented to the membership, it is printed herewith.

Minority Reports.

PHILADELPHIA, December 27th, 1905.

To the American Society of Civil Engineers,

GENTLEMEN:

In the report and specification of your Special Committee on Rails some points are conflicting and should be further discussed before a final report is made. It is therefore suggested that the present report be considered a report of progress, and presented to the Society for discussion at the Annual Convention in June, 1906.

The report makes a strong plea for lower temperatures in rolling in order to secure better metal in the rails, and every effort should be made to accomplish this without making too radical changes in the methods of manufacture. The high carbons called for in the specification are contrary to this idea, and are more in line with the practice of relying on the chemical composition alone to give hardness in the rails. There is no trouble in getting hardness in this way, but the rails are likely to be brittle, and thus increase the number of breakages. The large number of rails, irrespective of section and weight, which are now breaking in service, cannot be ignored, and, if necessary, it would be better to stand some additional wear, if rails could be secured which would not break.

In all Tee-rails the thin flanges control the finishing temperature, and, in order to get a rail which could be rolled at a lower temperature, it has been suggested to add more metal to the bottom of the flange of the present Am. Soc. C. E. section in order to allow a lower finishing temperature in rolling and retain the advantages of the present wide head for bearing surface and side wear. This would not prevent the metal in the head from breaking off in the plane of the web, as at present. This breakage occurs in all weights of rails, and is generally due to piped steel or segregation.

In order to prevent the heads from breaking, it has been suggested to use one standard width of head for rails of 80 lb. and more, increasing the thickness of the head and of the flange as the weight increases, thus producing a stiffer rail in the heavier sections, with a head better suited to withstand the severe effect of worn wheels riding the edge of the head. Rails of 80 to 100 lb. of such sections, could be finished at the same temperature in rolling, and therefore be made of the same carbon steel.

Notwithstanding the advantages which might be derived from such rails, it would be ill-advised to recommend any change of section until every means has been exhausted for producing better steel, on account of the large commercial interests involving both the manufacturer and consumer.

WM. R. WEBSTER.

Report of the Committee on Rail Sections (continued).

DECEMBER 28TH, 1905.

To the American Society of Civil Engineers,

GENTLEMEN:

While concurring in the major portion of the conclusions arrived at by the majority of your Special Committee on Rails, the undersigned deems it his duty to present a minority report.

Instruction No. 1.—He agrees with the majority report.

Instruction No. 2.—He agrees with the majority report with the following addition: The attention of the Society is called to the fact that certain failures in service, especially of rails of maximum sections, can be traced to the uneven temperatures of the various parts of the sections when finished at the rolls due to the large cross-section of head and relatively small areas of flange and web. This very dangerous defect can alone be remedied by a very radical change of design which your Committee, with the data now at their command, do not feel authorized in recommending, but believe that the attention of the Society should be called to the matter as a subject for future investigation and report.

Instruction No. 3.—He agrees with the majority report.

Instruction No. 4.—He dissents from the conclusion of the majority report. He is of the opinion that specifications as prepared by the user should extend only to the quality of the finished material both chemically and physically, the details and methods of manufacture being left entirely to the producer, but that the requirements be made so rigid that only material made in accordance with the best state of the art will fill the specifications. He dissents from the opinion:

"When rails of lighter sections were used, chemical conditions were not so important as with the heavier ones; this, because the rolling down of the steel to the smaller sections of necessity gave it more work, and also finished the rail at a lower temperature; thus producing a finer-grained and tougher metal. This partially explains why so many of the early steel rails gave good physical results, while their chemical composition was so irregular."

but believes that chemical composition was as important in the past as it is in the present, and equally so in all sections; the variations referred to can be accounted for by physical conditions rather than chemical composition.

As to other statements of the majority report under this clause he agrees.

Instruction No. 5.—He dissents from the clauses of the proposed specifications relating to methods of manufacture, and would omit the same. He agrees with those relating to quality and requirements of finished material, excepting as to carbon and manganese content.

He would also propose the addition of a test for tensile strength and elongation, the test piece to be cut from the head of the rail, and would add to the drop test a measure of limiting deflection.

He would also call attention to the following point; if a discard be required, it should be measured by a certain percentage of the ingot, and not by the length of the bloom.

The proposed method of regulating the finishing temperature will not necessarily accomplish the desired result, as the rail may be held prior to entering the finishing pass or at some other point until the proper finishing temperature be attained, while what is sought to be accomplished is that the ingot be worked throughout at a lower temperature than prevails in present practice.

He sees no reason why the carbon and manganese content, as specified for light sections, should be raised as the weight of the rails is increased, for the reason that the heavier the rail the more uneven the temperature at finishing throughout the various portions of the section, and the higher the carbon the more susceptible the steel to heat treatment, the heavier the rail the higher the temperature of the head at finishing; while, conversely, the higher the carbon the lower the working temperature should become. As long as we continue our present practice of using sections of such varying cross-sections as we have in our maximum weights, a high content of metalloids will always be a menace to successful service, which is not alone measured by wear of individual rails, but also by uniformity of results and freedom from sudden breakage.

His individual opinion is in favor of even a lower carbon content than proposed in the majority report, coupled with such physical requirements of finished material as will necessitate a rolling of the material at a lower temperature than now practiced.

Respectfully submitted,

PERCIVAL ROBERTS, JR.

To the American Society of Civil Engineers,

GENTLEMEN:

Referring to the preliminary report of the majority of the Special Committee on Rails, the writer considers that there are some doubts regarding various points, the first of which relates to the reaffirmation of the American Society of Civil Engineers' sections.

It is not understood from the synopsis of the reports received from the railroad companies, how many of those who express themselves as satisfied with the sections, are using rails of 80 and 85 lb. per yd. and upward to 100 lb. per yd. in weight.

Recent experience with heavier sections of rails, 85 lb. per yd. and greater in weight, has shown that there have been some troubles

Report of the Committee on Rail Sections (continued).

caused by the shearing or splitting of the heads of these heavy sections under severe traffic, and as comparatively few roads use the heaviest sections, the opinion of the majority could not govern, as most of the railways, when considered individually, use rails of the lighter weights from 85 lb. per yd. and less. It would appear that in forming an opinion of the results obtained with the heavy sections the different sizes should be considered separately and a careful valuation made of the experience, with due regard to the mileage, character and amount of traffic. To state this briefly, it is believed that a majority of favorable opinions on the sections, counting each railroad as one vote, regardless of the size of the road and sections used, does not answer the question regarding the heavier sections.

The above question of heavy sections, however, is believed to be not so great a one as that of the impossibility of obtaining low-phosphorus Bessemer rails, for the reason that a very serious and practical difficulty presents itself in that part of the specifications requiring that phosphorus shall not exceed 0.085 per cent. As, although the Lake Superior Districts produced about 30 000 000 tons of ore last year, there was not nearly enough of low-phosphorus grade to supply the demand for rails of such composition, and only a very small part of the required output might be made to the proposed specifications. This is a vital point, as it is certainly wrong to ask for something that cannot be had.

As regards Open-Hearth rails, it seems that the slight experience had with them up to this time leaves this question in a very immature condition.

Under these circumstances, the undersigned does not feel warranted in signing the committee's report, and believes that the subject should receive their further attention.

Geo. E. Thackray.

Discussion on Report of Committee on Rail Sections.

The President.—Gentlemen, you have heard the report of the Committee on Rail Sections. What action will you take?

Foster Crowell, M. Am. Soc. C. E.—I move that the report of the Committee be received, and also that the minority reports of the Committee be received for further consideration.

The President.—Gentlemen, one member of the Committee has asked that it be discharged, and one member requests that this report be referred to the Society for further discussion at the Annual Convention. Now, what is to be done?

Joseph O. Osgood, M. Am. Soc. C. E.—Is there any motion before the Society?

The President.—Yes, Mr. Crowell has made a motion.

Mr. Osgood.—Has that motion been seconded?

The President.—No.

Mr. Osgood.—Then I move that the report be received and the Committee continued.

The motion was seconded.

Robert W. Hunt.—As a member of that Committee, I suppose, from a strict parliamentary standpoint, if a committee does not finish its report, it should naturally be continued, but, at the same time, I do not think you can expect anything further, or any further work from this Committee. Eight out of the twelve members of this Committee have striven for four years to perform their duty, and to give you their best thought. Three of the members differed with us. It is not worth while to go into the details of the workings of that Committee, but, I think the members will agree with me, that we cannot hold out any hope that the Committee will be any closer together in June than it is now, and for that reason I think it is wise that the Society should not put itself on record as recommending the acceptance of the report of the Committee without careful thought. We will undoubtedly meet with great opposition on the part of the rail manufacturers of the country, and, while I believe it is very healthy to have that opposition, still the Society wants to be very careful in any position it takes.

I am confident that the status, so far as the Committee is concerned, will be the same next June as it is now, if it is continued in force.

Mr. Osgood.—I had in mind, when suggesting that the Committee should continue its work, that there was something more to be done, and if the Committee is discharged, it is a wrong process to appoint an entirely new Committee to take up the work; but, on the other hand, if this Committee is continued and any member feels that he cannot continue on the Committee, he can resign and another can be appointed in his place. I think the continuing of this old Committee and the appointment of new members to it, if necessary, will be better than the appointment of an entirely new Committee, to carry out the work if it is not already completed.

The President.—Are there any further remarks? Mr. Osgood, will you please repeat your motion?

Mr. Osgood.—My motion is that the report of the Committee be received and the Committee continued.

The President.—Are you ready for the question?

A Member.—Question.

The President.—Gentlemen, you have heard the motion. All in favor of Mr. Osgood's motion that the report be received and the Committee continued, say "aye."

("Ayes.")

The President.—Those opposed, "no."

(No response.)

Report of the Committee on Concrete and Reinforced Concrete.

The President.—The "ayes" have it, and the motion is carried. The report of the Special Committee on Concrete and Reinforced Concrete.

The Secretary of the Committee will please read the report.

The report was read by Mr. R. L. Humphrey, Secretary of the Committee, as follows:

REPORT OF THE SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE.

Mr. President and

Members of the American Society of Civil Engineers.

Gentlemen:

Your Special Committee on Concrete and Reinforced Concrete desires to present the following report:

The duty of your Committee is to formulate rules for the use of concrete and reinforced concrete.

It has been found, however, that the necessary data for this purpose are not available in cognate form, and it is necessary to carry on such investigations as will secure the requisite data.

The investigations being conducted at the various technological institutions have received very careful consideration, and, as a result, the Committee has concluded that the information to be derived from this source, while valuable, is not sufficiently conclusive for use in the formulation of a report, most of this information being only on special phases of the subject.

At a meeting of the Committee held in New York on October 11th, 1905, it was decided to form a number of sub-committees for the collation of existing literature and the results of previous investigations. These sub-committees are now at work, and it is expected that the results of their labors will be presented to the Society at its next Annual Meeting.

It was suggested that the work being undertaken by the United States Geological Survey would yield the additional data desired, and, at a meeting held in Cleveland, June 21st, 1905, the Committee considered this proposition and finally instructed several of its members to prepare a plan of co-operation for its consideration. At the meeting held in Atlantic City, June 30th, 1905, a plan of co-operation was presented, which, after a thorough discussion, was adopted.

At a meeting of the Committee held in New York, December 12th, 1905, the Sub-Committee on Tests presented a programme for such investigations as it deemed desirable to make, with subdivisions, which, with a few minor amendments, was adopted; this programme will be elaborated further, as the work progresses.

These investigations, which it is estimated will take at least five years, will be carried on in the laboratories of the United States

Geological Survey, involving an estimated yearly expenditure by the United States Government of upwards of $20 000.

At the same meeting the Sub-Committee on Ways and Means was instructed to raise $10 000 for the present year, for the purpose of defraying the expenses of the Committee in compiling the results of these investigations and for formulating a report.

The Committee now has its work well under way and has taken steps to secure the requisite funds for its expenses.

As rapidly as possible, the Committee will report such data or information as it considers of value.

Your Committee asks that it be continued.

Respectfully submitted, for the Committee,

C. C. Schneider,
Chairman;
Richard L. Humphrey,
Secretary.

Committee:

C. C. Schneider,
Richard L. Humphrey,
J. E. Greiner,
W. K. Hatt,
Olaf Hoff,
Robert W. Lesley,
J. W. Schaub,
Emil Swensson,
A. N. Talbot,
J. R. Worcester.

The President.—Gentlemen, you have heard the report. What will you do with it?

Morris R. Sherrerd, M. Am. Soc. C. E.—I move that the report be received and that the Committee be continued.

The motion, being duly seconded, was carried unanimously.

Amendments to the Constitution.

The President.—The amendments to the Constitution. The Assistant Secretary will please read the proposed amendments to the Constitution.

The Assistant Secretary read the following:

The following suggested amendments to the Constitution are submitted:

"Amend Article II, Section 2, as follows: insert after the word 'Civil' in the first line 'Engineer who shall have reached a position of recognized standing in the profession, in its several branches including.' Also strike out all of the second line beginning with the word 'Electrical' and insert 'and Electrical Engineering or in Architecture or Marine Architecture.' Also strike out all after the word 'age' in the fourth line.

Amendments to the Constitution (continued).

"The section will then read:

"2. A Member shall be a Civil Engineer who shall have reached a position of recognized standing in the profession, in its several branches, including Military, Naval, Mining, Mechanical and Electrical Engineering or in Architecture or Marine Architecture. He shall be at the time of admission to membership not less than thirty years of age."

The foregoing amendments are proposed by the following Corporate Members: Jas. Owen, Ralph H. Chambers, Philip W. Henry, Walter H. Sears, S. Whinery, A. P. Boller, Foster Crowell.

The President.—Some letters on the subject have been received, and these will be read by the Assistant Secretary.

The Assistant Secretary then read letters from Messrs. Charles H. Ledlie, L. S. Randolph, Dugald C. Jackson, Benjamin Thompson, Edward M. Boggs, Ellis B. Noyes, Joseph Lillich, A. B. Wood, Robert L. Lund and Mark L. Ireland.

Discussion on Amendments to the Constitution.

S. Whinery, M. Am. Soc. C. E.—In order to get this matter formally and in proper shape before the Society, I want to say that there seems to be quite a strong conviction among the membership that a part of the Constitution ought to be changed, in some way, but there does not seem to be much unanimity as to what these changes should be.

It is very difficult, in a meeting like this, to give such a subject proper attention. I think that in dealing with a question of this kind the best results can be obtained by appointing a carefully chosen Committee, who can take up the subject and give it all the attention necessary, and report to the Society.

I do not wish to cut off further discussion of the subject at this time, but wish to say that I shall, at the proper time, move for adoption, that the amendment be referred to a committee of five members to be appointed by the President, which committee shall report to the next Annual Convention, what, if any, changes it deems advisable, in the present classification of the membership of the Society, or any modification in each grade of membership, and submit any amendment to these amendments which they may recommend.

I will not move the adoption of that resolution now, because it may have the effect of cutting off any discussion that we might have on it here to-day, which would be valuable to the committee. I think the discussion on the subject should proceed now, and at the proper time I will move the adoption of that motion.

Mr. Crowell.—I think this subject is of extreme importance to the Society and its work. This is not the first time, by any means, that the subject has been brought forward, but I think the letters which have been read are indicative of the fact that the subject has become of very vital importance. When Mr. Whinery gets ready to

offer his motion I will be very glad to second it. I do not see why the adoption of that motion should come after the discussion. In fact, I do not think it is worth while to spend much time now in discussing the question, for the reason that Mr. Whinery has just stated—that justice cannot be done to such a subject in a meeting of this kind. It requires more thought, and requires comparison with the various standards of other societies and other organizations, and with our own past history. I think it would be well for Mr. Whinery to offer his motion now, and then any discussion that seems proper can be had upon it.

The President.—Will you offer that motion now, Mr. Whinery?

Mr. Whinery.—If it will not have the effect of ending the discussion, I will offer it.

The President.—Discussion on that motion will then be in order.

Mr. Crowell.—I second the motion.

The President.—In order to have the motion understood, will you please repeat it?

The Assistant Secretary read the motion, as follows:

"Moved, that the amendments be referred to a committee of five Corporate Members, to be appointed by the President, which committee shall report to the next Annual Convention what, if any, changes they deem advisable in the present classification of the membership of the Society, or in the qualifications for each grade of membership; and submit any amendments to these amendments which they may recommend."

The President.—Discussion on this motion is now in order. Are there any remarks?

A Member.—Question.

Edward P. North, M. Am. Soc. C. E.—Mr. President, I would like to offer an amendment to that motion, and have it seconded: That the five Corporate Members mentioned in the motion be increased by one Associate and one Junior. I think that, in the consideration of such a question, these two elements are quite important classes in our Society, and should be represented.

The amendment was accepted.

The President.—You have heard the motion, gentlemen. Those in favor of it say "aye."

The motion as amended was carried unanimously.

The President.—The appointment of members of the Nominating Committee for each of the seven geographical districts, to serve for two years, is now in order. The Assistant Secretary will please read the nominations. **Nominating Committee.**

The Assistant Secretary.—I will first read the nominations for District No. 1.

Nominating Committee (continued).

District No. 1.—Total number of votes received, 90; distributed as follows:

John H. Cook	37
Alfred Noble*	2
Clemens Herschel	2
George B. Francis	2
A. P. Boller	2
Edlow W. Harrison	2
C. L. Harrison*	2
O. F. Nichols	2
J. V. Davies	2

The following have received one vote each:

Henry R. Asserson,
Josiah A. Briggs,
C. W. Buchholz,
R. S. Buck,
George Hallett Clark.
D. C. N. Collins,
Alfred Craven,*
J. J. R. Croes,*
J. H. Edwards,
Robert Giles,
Charles H. Graham,
George A. Harwood,
William J. Haskins,
Philip W. Henry,
R. Hering,
William R. Hill,
Henry W. Hodge,
Otis E. Hovey,
Joseph Mayer,
William C. Merryman,
Rudolph P. Miller,
Mace Moulton,
Charles H. Myers,
Frederick S. Odell,
C. J. Parker,
C. D. Pollock,
William A. Pratt,
Robert Ridgway,
Frank W. Skinner,
Frank J. Sprague,
S. C. Thompson,
R. H. Tingley,
Henry Vier,
John C. Wait,
Clement I. Walker,
Charles D. Ward,
Samuel Whinery.

On motion, duly seconded, John H. Cook, Assoc. M. Am. Soc. C. E., was appointed a member of the Nominating Committee for the First District.

The Assistant Secretary.—The nominations for District No. 2 are as follows:

District No. 2.—Total number of votes received, 51; distributed as follows:

Leonard Metcalf	26
J. R. Worcester	6
Hiram A. Miller	3
Edwin D. Graves	2

* Ineligible.

The following have received one vote each:

C. F. Allen,	A. B. Hill,
Dexter Brackett,	F. W. Hodgdon,
Robert A. Cairns,	William E. McClintock,
Otis F. Clapp,	Henry Manley,
F. S. Curtis,	William Harley Moore,
Alexis H. French,	Charles W. Sherman,
Richard A. Hale,	E. K. Turner.

On motion, duly seconded, Leonard Metcalf, M. Am. Soc. C. E., was appointed a member of the Nominating Committee for the Second District.

The Assistant Secretary.—The nominations for District No. 3 are as follows:

District No. 3.—Total number of votes received, 69; distributed as follows:

A. H. Sutermeister	37
James H. Edwards	12
Phelps Johnson	4
William A. Haven	3

The following have received one vote each:

W. A. Brackenridge,	Olin H. Landreth,
E. R. Cary,	P. A. Peterson,
A. P. Davis,	George A. Ricker,
W. T. Jennings,	E. Gybbon Spilsbury,
Wallace C. Johnson,	Charles F. Stowell,
John Kennedy,	A. H. Van Cleve,
J. Van der Hoek.	

On motion, duly seconded, A. H. Sutermeister, M. Am. Soc. C. E., was appointed a member of the Nominating Committee for the Third District.

The Assistant Secretary.—The nominations for District No. 4 are as follows:

District No. 4.—Total number of votes received, 100; distributed as follows:

Edgar Marburg	41
Percival M. Sax	19
William Copeland Furber	9
D. D. Carothers	3
Thomas H. Johnson	3
Thomas W. Symons	2

Nominating Committee (continued).

The following have received one vote each:

James Murray Africa,	Frederic C. Kunz,
Kenneth Allen,	J. K. Lyons,
J. C. Bland,	Alexander Mackenzie,
John N. Chester,	D. E. McComb,
Mendes Cohen,	E. K. Morse,
Arthur P. Davis,	F. H. Newell,
Paul Didier,	Robert Van A. Norris,
B. T. Fendall,	H. H. Quimby,
C. E. Grunsky,	L. Y. Schermerhorn,
L. M. Haupt,	William L. Sibert,
Morris Knowles,	Emil Swensson,*

Frank Wilcox.

On motion, duly seconded, Edgar Marburg, M. Am. Soc. C. E., was appointed a member of the Nominating Committee for the Fourth District.

The Assistant Secretary.—The nominations for District No. 5 are as follows:

District No. 5.—Total number of votes received, 105; distributed as follows:

John W. Alvord	29
Gardner S. Williams	26
Willard Beahan	15
H. G. Kelley	9
Joseph Ripley	2

The following have received one vote each:

Onward Bates,*	Ralph Modjeski,*
W. L. Breckenridge,	Augustus Mordecai,
Charles C. Brown,	Frank C. Osborn,
Theodore L. Condron,	Isham Randolph,
William de la Barre,	James Ritchie,
Malverd A. Howe,	L. W. Rundlett,
Clarence W. Hubbell,	J. W. Schaub,
H. G. Kelley,	Harry E. Talbott,
Jesse Lowe,	W. D. Taylor,
Daniel McCool,	Gaylord Thompson,
Benjamin McKeen,	C. A. Wilson,
Dabney H. Maury,	George Y. Wisner.

On motion, duly seconded, John W. Alvord, M. Am. Soc. C. E., was appointed a member of the Nominating Committee for the Fifth District.

* Ineligible.

THE ASSISTANT SECRETARY.—The nominations for District No. 6 are as follows:

District No. 6.—Total number of votes received, 64; distributed as follows:

IRA G. HEDRICK	12
A. H. ZELLER	12
W. W. COE	3
J. F. COLEMAN	2
GEORGE G. EARL	2
WILLIAM H. COURTENAY	2
JULIAN W. KENDRICK	2
J. F. HINCKLEY	2
HUNTER McDONALD*	2
A. M. SCOTT	2

The following have received one vote each:

DANIEL BONTECOU,
STEPHEN P. BROWN,
C. S. BURNS,
W. E. GUNN,
E. A. HARPER,
B. M. HARROD,
J. N. HAZLEHURST,
W. B. W. HOWE,
CHARLES HUMPHREYS,
J. L. LUDLOW,
ROBERT MOORE,*
MARSHALL MORRIS,
E. T. D. MYERS,
J. A. OCKERSON,
HENRY B. RICHARDSON,
S. BENT RUSSELL,
F. W. SCARBOROUGH,
J. E. SIRRINE,
H. R. STANFORD,
HENRY M. STEELE,
C. A. WENTWORTH,
C. H. WEST,
NISBET WINGFIELD.

On motion, duly seconded, A. H. Zeller, Assoc. M. Am. Soc. C. E., was appointed a member of the Nominating Committee for the Sixth District.

THE ASSISTANT SECRETARY.—The nominations for District No. 7 are as follows:

District No. 7.—Total number of votes received, 57; distributed as follows:

FRANK O. MARVIN	16
FRANKLIN RIFFLE	4
ARTHUR L. ADAMS	3
R. B. MARSHALL	2
ROBERT B. BURNS	2

* Ineligible.

Nominating Committee (continued).

The following have received one vote each:

Jeremiah Ahern,	J. B. Lippincott,
D. D. Clarke,	M. L. Lynch,
William E. Dauchy,	F. B. Maltby,
W. W. Follett,	S. D. Mason,
Edward Gillette,	William G. Raymond,
R. H. Gresham,	G. H. Robinson,
C. E. Grunsky,	D. W. Ross,
W. W. Harts,	W. H. Sanders,
John B. Hawley,	James D. Schuyler,
F. W. D. Holbrook,	H. A. Sumner,
C. T. Johnston,	T. U. Taylor,
W. H. Jones,	A. J. Wiley.
A. E. Kastl,	Charles F. Williams,
A. V. Kellogg,	C. B. Wing,
Lewis Kingman,	E. T. Wright.

On motion, duly seconded, Frank O. Marvin, M. Am. Soc. C. E., was appointed a member of the Nominating Committee for the Seventh District.

The President.—Has the Secretary any announcements to make?

Award of Prizes.

The Assistant Secretary.—I have to report that the Board of Direction has awarded the prizes for the year ending July, 1905, in accordance with the report of the Committee appointed to recommend the awards, as follows:

Philadelphia, Penna., Dec. 26th, 1905.

To the Board of Direction,
American Society Civil Engineers.

Gentlemen:—Your Committee appointed for the purpose, recommend the award of prizes for papers, published during the year ending July 1, 1905, as follows:

The Norman Medal to Mr. C. C. Schneider, for Paper No. 997, "The Structural Design of Buildings."

The Thomas Fitch Rowland Prize to Messrs. Charles L. Harrison and Silas H. Woodard, for Paper No. 981, "Lake Cheesman Dam and Reservoir."

The Collingwood Prize for Juniors to Mr. E. P. Goodrich, for Paper No. 983, "Lateral Earth Pressures and Related Phenomena."

James Christie,
J. T. Fanning,
F. C. Kunz,
Committee.

Announcements.

The Thirty-eighth Annual Convention of the Society will be held at the Hotel Frontenac, Thousand Islands, New York, June 26th to 29th, 1906.

I have to announce the death of Francis Edward Snyder, elected Member September 6th, 1905; died December 23d, 1905.

The Assistant Secretary also made some announcements relative to the programme of the Annual Meeting.

The President.—We will now take an intermission until such time as the tellers' report is ready to be submitted.

(Recess.)

The President.—(After recess.) Gentlemen, the report of the tellers is ready.

The Assistant Secretary.—The tellers report:

Report of Tellers.

Report of Tellers Appointed to Canvass the Ballot for the Election of Officers at the Annual Meeting, January 17th, 1906.

Total number of ballots received	845
Defective	24
Total number counted	821

For President:

Frederic P. Stearns	817
Josiah A. Briggs	1
E. L. Corthell	1
Fayette S. Curtis	1

For Vice-Presidents:

Onward Bates	811
Bernard R. Green	801
H. G. Kelley	1
A. C. Cunningham	1
Almon B. Atwater	1
John A. Ockerson	1
E. C. Shankland	1
George S. Webster	1

For Treasurer:

Joseph M. Knap	815
John Thomson	1

For Directors:

George Gibbs	802
J. Waldo Smith	808
Emil Swensson	796
James M. Johnson	802
Wynkoop Kiersted	799
William B. Storey, Jr.	800
E. Marburg	1

Report of Tellers (continued).

E. M. Scofield	1
John Sterling Deans	2
Arthur P. Davis	1
A. L. Johnson	1
L. W. Rundlett	1
L. L. Tribus	1
J. F. Coleman	2
James Murray Africa	1
Arthur L. Adams	2
William G. Wilkins	2
James H. Brace	1
W. W. Coe	1
B. M. Wagner	1
Frank O. Marvin	1
E. B. Cushing	1
A. V. Kellogg	1
William G. Brenneke	1
George E. Gifford	1
A. B. Corthell	1
William H. Wiley	1
Ashbel E. Olmsted	1
E. Wegmann	1
Arthur Hider	1
J. A. Omberg, Jr	1
H. H. McClintock	1
George S. Greene, Jr	1
J. A. Atwood	1
Robert A. Cummings	1
J. A. L. Waddell	1
Thomas W. Symons	1
E. J. Beard	1
George B. Francis	1

W. D. Kelley,
Geo. L. Wilson,
J. A. Knighton,
Tellers.

Officers Elected.

The President.—The following having received a majority of the votes cast are herewith declared to be elected as officers of this Society for the ensuing year:

As President, Frederic P. Stearns. (Applause.)

As Vice-Presidents, Onward Bates and Bernard R. Green;

As Treasurer, Joseph M. Knap;

As Directors, George Gibbs, J. Waldo Smith, Emil Swensson, James M. Johnson, Wynkoop Kiersted and William B. Storey, Jr.

The Chair will appoint Mr. Green and Mr. Swensson as a committee to conduct Mr. Stearns to the platform. (Applause.)

President Stearns Introduced.

The President.—I take pleasure in introducing to you Mr. Stearns, President-elect of the American Society of Civil Engineers. (Applause.)

Remarks by President Stearns.

President Stearns.—Gentlemen of the American Society of Civil Engineers, I thank you for your cordial greeting.

Permit me to express to you my grateful appreciation of the high honor that you have conferred upon me in electing me as President of this Society which we all esteem.

The mere mention of the name—The American Society of Civil Engineers—brings with it suggestions of a Society which stands for the maintenance of the highest professional standing among its members; of a Society whose members accomplish marvels in these days of progress; of a Society rapidly growing both in numbers and resources, and capable of accomplishing great results, such as were produced by the International Engineering Congress; it brings with it also the memories of those who are not with us, eminent and honored members who were devoted to its interests.

In accepting the Presidency and the responsibility which goes with it, I recognize my own limitations; but, with your co-operation, I will try to achieve the best measure of success that lies within my power.

I thank you once more, gentlemen, for the honor conferred. (Applause.)

Adjourned.

The meeting then adjourned.

EXCURSIONS AND ENTERTAINMENTS AT THE FIFTY-THIRD ANNUAL MEETING.

Wednesday, January 17th, 1906.—After the business meeting, lunch for about 500 members was served at 1.30 P. M. at the Society House.

At 3 P. M., excursions under special guidance were organized, and visited the following engineering works: The Fifty-ninth Street Power House of the Interborough Rapid Transit Company, George H. Pegram, M. Am. Soc. C. E., Chief Engineer; the Blackwell's Island Bridge, O. F. Nichols, M. Am. Soc. C. E., Chief Engineer; the Pennsylvania Railroad Terminal Station, George C. Clarke, M. Am. Soc. C. E., Resident Engineer; the Waterside Stations of the New York Edison Company, Thirty-eighth to Fortieth Streets and East River, Thomas F. Murray, General Manager.

At 9 P. M. there was a Reception, with dancing, in the Society House, at which the attendance was about 350.

Thursday, January 18th, 1906.—The day was devoted to an excursion, by steamer, to the New York Navy Yard, by invitation of Rear-Admiral Joseph B. Coghlan, Commandant.

The party met at the pier at the foot of West Forty-first Street (North River) at 10 A. M., and embarked on the steamer *Valley Girl*, kindly furnished through the courtesy of General Howard Carroll and the Hon. John H. Starin. The vessel proceeded down the North River, around the Battery, and directly to the Navy Yard, where the party was received by the Commandant and officers in charge. All the buildings, dry docks and other features of the Yard were open for inspection, and quite a large number visited the United States Battleship *Connecticut*, which was being fitted out.

On leaving the Navy Yard the steamer proceeded up the East River as far as Riker's Island, thus affording a view of the various bridges, and incidentally other engineering works on the river front. Lunch was served on the boat, and the party landed at the Recreation Pier at the foot of East Twenty-fourth Street at about 5 P. M.

In the evening, at the Society House, Charles S. Gowen, M. Am. Soc. C. E., presented a paper entitled, "The Changes at the New Croton Dam," which was illustrated with lantern slides, and the subject was discussed by W. R. Hill, M. Am. Soc. C. E. This was followed by a social and informal "smoker," at which there was an attendance of more than 500 members and guests.

The following list contains the names of 605 members, of various grades, who registered as being in attendance at the Annual Meeting. The list is incomplete, on account of the failure of many members to register, and it does not contain the names of any of the guests of the Society or of individual members:

Abbot, F. W....New York City
Abbott, H......New York City
Aiken, W. A......Pittsburg, Pa.
Alderman, C. A., Worcester, Mass.
Allen, C. H.....New York City
Allen, C. L........Utica, N. Y.
Allen, E. Y., South Orange, N. J.
Allen, H. C.....Syracuse, N. Y.
Allen, W. A.....New York City
Andrews, Horace..Albany, N. Y.
Ash, Dorsey...Wilmington, Del.
Aspinwall, T......Boston, Mass.
Atkinson, A....New York City
Auryansen, F..Brooklyn, N. Y.
Averill, F. L..Washington, D. C.

Babcock, W. S..New York City
Backes, W. J..Weehawken, N. J.
Bacon, John W.Danbury, Conn.
Bailey, G. I....New York City
Baird, H. C.....New York City
Baker, H. C., Jr.New York City
Baldwin, F. H..Brooklyn, N. Y.
Baldwin, W. J..New York City
Ball, E. S......New York City
Bamford, W. B..New York City
Bardol, F. V. E..Buffalo, N. Y.
Barney, Percy C.Brooklyn, N. Y.
Bartoccini, A....New York City
Bartram, G. C....Boston, Mass.
Basinger, J. G...New York City
Beerbower, G. M.New York City
Belden, E. T..New Haven, Conn.
Belknap, J. M..Ashtabula, Ohio
Belknap, W. E..New York City
Bell, T. K....Philadelphia, Pa.
Belzner, T......New York City
Benton, L. S....New York City
Berg, Walter G..New York City
Berger, Bernt...New York City
Bernegau, C. M..New York City
Berry, G.......Brooklyn, N. Y.
Betts, R. T......New York City
Beugler, E. J...New York City

Bishop, G. H.Middletown, Conn.
Bishop, H. K....Hudson, N. Y.
Bissell, H........Boston, Mass.
Blakeley, G. H..Paterson, N. J.
Boardman, H. E.New York City
Boecklin, Werner.New York City
Bogart, John....New York City
Boller, A. P.....New York City
Bond, G. W., Jr., Weehawken, N. J.
Boucher, W. J..New York City
Bouton, H....Jersey City, N. J.
Bowman, A. L...New York City
Boyd, J. C......New York City
Brace, J. H.....New York City
Brackenridge, J. C., Brooklyn, N. Y.
Braine, L. F....New York City
Bramwell, G. W.New York City
Braslow, Barnett.New York City
Breuchaud, J....Yonkers, N. Y.
Breuchaud, J. R..Yonkers, N. Y.
Briggs, R. E....Mexico, Mexico
Brinckerhoff, H. W., New York City
Brown, L. L....New York City
Brown, S. C....New York City
Brown, T. E....New York City
Brown, W. P....New York City
Bruce, J. M....New York City
Bryson, Andrew.New Castle, Del.
Buchholz, C. W..New York City
Buck, L. L.....New York City
Buck, R. S......New York City
Bullock, W. D..Providence, R. I.
Burgess, G. H..New York City
Burr, W. H.....New York City
Burrowes, H. G..New York City
Bush, H. D.....Baltimore, Md.
Bush, L.....East Orange, N. J.
Buzzell, J. W.....Boston, Mass.

Cantine, E. I.East Orange, N. J.
Carlile, T. J....New York City
Carpenter, A. W.New York City

Carter, A. E....New York City
Chambers, H. J..New York City
Chapleau, S. J.....Ottawa, Ont.
Chester, J. N.....Pittsburg, Pa.
Christian, G. H.New York City
Christy, G. L....New York City
Clapp, S. K.....New York City
Clark, G. Hallett.New York City
Clark, J. A., Jr.,
East Orange, N. J.
Clarke, G. C....New York City
Clarke, St. John...Bogota, N. J.
Coe, W. W........Roanoke, Va.
Coffin, Amory...New York City
Coffin, T. Amory.New York City
Cole, H. O......New York City
Collier, B. C....New York City
Collingwood, F..Elizabeth, N. J.
Conger, A. A....Albany, N. Y.
Cook, J. H......Paterson, N. J.
Coombs, S. E....New York City
Cooper, S. L....Yonkers, N. Y.
Cornell, G. B....New York City
Cornell, J. N. H.New York City
Corthell, A. B...New York City
Cotton, J. P.....Newport, R. I.
Coverdale, W. H.New York City
Crane, C. A.....New York City
Crane, J. S......Newark, N. J.
Craven, Alfred..New York City
Crehore, W. W..New York City
Cresson, B. F., Jr.,
New York City
Creuzbaur, R. W.New York City
Crowell, Foster..New York City
Cuddeback, A. W.Paterson, N. J.
Cummings, Noah.New York City
Curtis, F. S......Boston, Mass.
Curtis, V. P...Worcester, Mass.

Daily, J. A..East Orange, N. J.
Dakin, A. H., Jr.New York City
Dalrymple, F. W.Bayonne, N. J.
Davies, J. V....New York City
Davis, A. L.....New York City
Davis, C........Edgeworth, Pa.
Davis, J. L......New York City
Davis, R. B......Boston, Mass.
Davis, W. R......Albany, N. Y.
Dean, Luther....Taunton, Mass.
Deans, J. S....Phœnixville, Pa.
Decker, J. H. Atlantic City, N. J.
Devin, G.......New York City
Diamant, A. H..New York City
Dilks, L. C......New York City
Dimon, D. Y....New York City
Dorrance, W. T..New York City
Doty, A. Duane..New York City
Dougherty, R. E.New York City
Downes, A. K...New York City
Drew, C. D.....Brooklyn, N. Y.
Dunham, H. F..New York City
Dunlap, F. C..Philadelphia, Pa.

Easby, M. W..Philadelphia, Pa.
Edwards, D. G..New York City
Edwards, H. W..New York City
Edwards, J. H....Oxford, N. Y.
Ellis, J. W....Woonsocket, R. I.
Elwell, C. C.New London, Conn.
Evans, L. H....New York City

Farrington, H...Yonkers, N. Y.
Fenton, L. G....New York City
Fetherston, J. T.,
New Brighton, N. Y.
Fickes, E. S......Pittsburg, Pa.
Fisher, E. A...Rochester, N. Y.
Fisher, F. D....New York City
Fisher, H. T....New York City
Fisher, W.....Philadelphia, Pa.
Fletcher, R.....Hanover, N. H.
Flinn, Alfred D..New York City
Forchhammer, H.New York City
Ford, P. D......New York City
Ford, W. H...Philadelphia, Pa.
Forgie, James...New York City
Fort, E. J......New York City
Foster, E. H....New York City
Francis, G. B....New York City

Francis, H. N. . Providence, R. I.
Fraser, C. E. . . . . New York City
French, A. W. . Worcester, Mass.
French, J. B. . . . Jamaica, N. Y.
Frick, Walter. . . . Lewisburg, Pa.
Frost, G. S. . . . . Brooklyn, N. Y.
Fuller, F. L. . . . . . Boston, Mass.
Fuller, W. B. . . . . Pittsburg, Pa.
Fuller, W. E. . . . New York City
Furber, W. C. . Philadelphia, Pa.

Gahagan, W. H. Brooklyn, N. Y.
Gartensteig, C. . . New York City
Gaston, L. P. . . . New York City
Gifford, G. E. . . . New York City
Giles, A. L. . . . Providence, R. I.
Giles, Robert. . . Lawrence, N. Y.
Gillen, W. J. . Croton Falls, N. Y.
Gillespie, R. H. . New York City
Gillette, H. P. . . New York City
Goad, C. E. . . . Toronto, Canada
Goldsborough, J. B.,
Croton-on-Hudson, N. Y.
Goodrich, E. P. . New York City
Goodridge, J. W. New York City
Gould, J. W. duB. New York City
Gould, W. T. . . . New York City
Gow, C. R. . . . . Boston, Mass.
Gowen, C. S. . . . Ossining, N. Y.
Gowen, S. . . . . . . Phœnixville, Pa.
Grady, C. B. . West Orange, N. J.
Graham, C. H. . New York City
Graham, J. M. . . New York City
Grant, T. H. . . . Red Bank, N. J.
Gray, J. C. . . . . . Brooklyn, N. Y
Gray, J. H. . . . . . New York City
Gray, W. . . . . New York City
Green, B. R. . Washington, D. C.
Green, C. N. . . . . New York City
Green, F. M. . . . . New York City
Greene, Carleton. New York City
Greene, G. S., Jr. New York City
Greenlaw, R. W. . New York City
Gregory, C. E. . . New York City
Greiner, J. E. . . . Baltimore, Md.
Griffin, A. J. . . . Brooklyn, N. Y.
Grimes, E. L. . . . . . . Troy, N. Y.
Grimm, C. R. . . . New York City
Guthrie, K. O. . Brooklyn, N. Y.

Hager, A. B. . Rutherford, N. J.
Haight, S. S. . . . New York City
Haines, C. W. . . Richmond, Va.
Hale, H. M. . . . . . New York City
Hall, L. W. . . . . Syracuse, N. Y.
Hall, M. W. . . . . New York City
Hallsted, J. C. . . New York City
Hanavan, W. L. . New York City
Harrison, C. L. . New York City
Harrison, E. W.,
Jersey City, N. J.
Harte, C. R. . New Haven, Conn.
Harwood, G. A. . New York City
Haskins, W. J. . New York City
Hauck, W. . . . . . Brewster, N. Y.
Hayes, H. W. . . . . Boston, Mass.
Hazard, S. . White Plains, N. Y.
Hazelton, C. W.,
Turners Falls, N. Y.
Hazen, Allen. . . . New York City
Hazen, John V. . Hanover, N. H.
Hazen, W. N. . . . New York City
Hazlett, R. . . Wheeling, W. Va.
Heald, E. C. . Washington, D. C.
Herbert, H. M.,
Bound Brook, N. J.
Hering, Rudolph. New York City
Hermanns, F. E. . New York City
Herschel, C. . . . . New York City
Hewes, Virgil H. New York City
Hewitt, C. E. . . . Trenton, N. J.
Hildenbrand, W. . New York City
Hill, A. B. . . New Haven, Conn.
Hill, G. . . . . New York City
Hill, W. R. . . . . . Brooklyn, N. Y.
Hillyer, W. R.,
West Brighton, N. Y.
Hoag, S. W., Jr. . New York City
Hodgdon, F. W. . . Boston, Mass.
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Hogan, J. P.... New York City
Holbrook, Percy. New York City
Hollyday, R. C.. New York City
Honness, G. G... Katonah, N. Y.
Hood, R. H..... New York City
Horton, S...... Garrison, N. Y.
Hough, D. L.... New York City
Hovey, O. E.... New York City
How, R. W...... New York City
Howard, E. E.. Kansas City, Mo.
Howard, J. L..... Boston, Mass.
Howe, H. J...... New York City
Howell, R. P.. Philadelphia, Pa.
Howes, D. W.... New York City
Hoyt, W. E.... Rochester, N. Y.
Hubbell, G. S.... New York City
Hudson, C. W.. Phœnixville, Pa.
Hughes, H. J.. Cambridge, Mass.
Humphrey, R. L., Philadelphia, Pa.
Hunt, C. A.... Brooklyn, N. Y.
Hunt, Robert W... Chicago, Ill.
Huntington, G. D., Watertown, N. Y.

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Jacobs, R. H.... New York City
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Janvrin, N. H... New York City
Japp, H. Long Island City, N. Y.
Johnson, A. L.... St. Louis, Mo.
Johnson, L. J.. Cambridge, Mass.
Jonson, E. F.... New York City
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Just, G. A....... New York City

Kastl, A. E...... Clinton, Mass.
Keays, R. H..... New York City
Keith, H. C...... New York City
Keller, C. L....... Chicago, Ill.
Kelley, W. D.... Yonkers, N. Y.
Kenly, W. W.... New York City
Kenney, E. F.. Philadelphia, Pa.
King, C. C., West New Brighton, N. Y.
King, P. S...... New York City
King, W., Jr... New York City
Kinsley, T. P.... New York City
Kirchner, P. A.. New York City
Knap, Edgar D.. New York City
Knap, Joseph M. New York City
Knickerbocker, J... Troy, N. Y.
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Knighton, J. A.. New York City
Knowlton, T.... New York City
Kolb, H. J..... Brooklyn, N. Y.
Kuichling, E.... New York City

LaChicotte, H. A. New York City
Lamb, Richard.. New York City
Landreth, O. H., Schenectady, N. Y.
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Lee, W. B..... Hillburn, N. Y.
Leffingwell, F. D. Montclair, N. J.
Leisen, T. A.. Wilmington, Del.
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Lesley, R. W.. Philadelphia, Pa.
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Lewis, N. P..... New York City
Lieb, J. W., Jr.. New York City
Lindenthal, G.... New York City
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Long, E. McL... New York City
Look, M. J..... Katonah, N. Y.
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Merrill, O. . . . . . New York City
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Swindells, J. S.Mt. Kisco, N. Y.

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Thomson, G. H..New York City
Thomson, John..New York City
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Tompson, G. M...Boston, Mass.
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Tribus, L. L....New York City
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Trotter, A. W...New York City
Trout, C. E.....New York City
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Tucker, W. C...New York City
Tull, R. W.....New York City
Turner, D. L....New York City
Turner, E. K.....Boston, Mass.
Tuttle, A. S....New York City

Ulrich, Daniel..Katonah, N. Y.
Usina, D. A....New York City

Vail, J. J.......Rahway, N. J.
Van Brakle, J. O., New York City
Van Buren, J. D., New Brighton, N. Y.
Van Buskirk, C. R., New York City
Van Suetendael, A. O., Yonkers, N. Y.
Verveer, E. L...New York City
Vier, Henry....New York City
Vinton, T. MacI..New York City
Vorce, C. B.....New York City
Vredenburgh, W., Jr., New York City

Wachter, C. L..New York City
Wadsworth, J. E., New York City
Wagner, J. C..Philadelphia, Pa.
Wagner, S. T..Philadelphia, Pa.
Waldron, S. P..New York City
Walker, C. I....New York City
Walker, J. W....Pittsburg, Pa.

Wallace, H. U...New York City
Ward, C. R......New York City
Wason, L. C.....Boston, Mass.
Waterhouse, J..New York City
Watkins, F. W.,
White Plains, N. Y.
Watson, Merritt.New York City
Webster, W. R..Philadelphia, Pa.
Welton, B. F...New York City
Wendt, E. F.....Pittsburg, Pa.
West, C. H....Greenville, Miss.
West, O. J........Chicago, Ill.
Weston, R. S.....Boston, Mass.
Weymouth, F. E.,
Glendive, Mont.
Wheeler, H. R..New York City
Wheeler, R. N..New York City
Whinery, Samuel.New York City
White, L.......New York City
White, W. M....New York City
Wiggin, E. W.New Haven, Conn.
Wilcock, F......Brooklyn, N. Y.
Wilcox, F........Pittsburg, Pa.
Wild, H. J.....Meriden, Conn.
Wilgus, W. J....New York City
Wilkes, J. K.,
New Rochelle, N. Y.
Wilkins, G. S...New York City
Williams, B. L., Jr.,
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Williamson, F. S.,
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Wills, A. J...Newburgh, N. Y.
Wilson, C. W. S..New York City
Wilson, G. L.,
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Woodbury, C. J. H.,
Boston, Mass.
Woodcock, H. W.,
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Washington, D. C.
Wright, A. W....Pomona, Cal.
Wright, J. B....New York City

Yates, E. A.....New York City
Yates, W. H.....New York City
Young, E. E..Auburndale, Mass.

Zollinger, L. R.,
Philadelphia, Pa.

ANNOUNCEMENTS.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

MEETINGS.

Wednesday, March 7th, 1906.—8.30 P. M.—A regular business meeting will be held. Ballots for membership will be canvassed, and a paper, entitled "The Theory of Continuous Columns," by Ernst F. Jonson, Assoc. M. Am. Soc. C. E., will be presented for discussion.

This paper was printed in *Proceedings* for January, 1906.

Wednesday March 21st, 1906.—8.30 P. M.—At this meeting a paper, entitled "New Facts about Eye-Bars," by Theodore Cooper, M. Am. Soc. C. E., will be presented for discussion.

This paper was printed in *Proceedings* for January, 1906.

Wednesday, April 4th, 1906.—8.30 P. M.—A regular business meeting will be held. Ballots for membership will be canvassed, and a paper, entitled "The Panama Canal," by A. G. Menocal, M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings.*

Wednesday, April 18th, 1906.—8.30 P. M.—At this meeting a paper, entitled "A Complete Analysis of General Flexure in a Straight Bar of Uniform Cross-Section," by L. J. Johnson, M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings.*

Wednesday, May 2d, 1906.—8.30 P. M.—A regular business meeting will be held. Ballots for membership will be canvassed, and a paper, entitled "The Control of Hydraulic Mining in California by the Federal Government," by William W. Harts, M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings.*

ANNUAL CONVENTION.

The Thirty-eighth Annual Convention of the Society will be held at The Hotel Frontenac, Thousand Islands, New York, on June 26th to 29th, 1906.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all meetings:

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Society of Engineers, 17 Victoria Street, Westminster, S. W., England.

American Institute of Mining Engineers, 99 John Street, New York City.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Civil Engineers' Club of Cleveland, 1200 Scofield Building, Cleveland, Ohio.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Philadelphia, 1122 Girard Street, Philadelphia, Pa.

Engineers' Society of Western Pennsylvania, 410 Penn Avenue, Pittsburg, Pa.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Engineers' Club of Central Pennsylvania, Corner, Second and Walnut Streets, Harrisburg, Pa.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Teknisk Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Société des Ingénieurs Civils de France, 19 Rue Blanche, Paris, France.

Svenska Teknologförenigen, Brunkebergstorg 18, Stockholm, Sweden.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Sachsischer Ingenieur- und Architekten- Verein, Dresden, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Pacific Northwest Society of Engineers, 617–618 Pioneer Building, Seattle, Wash.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Memphis Engineering Society, Memphis, Tenn.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

The Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Cleveland Institute of Engineers, Middlesbrough, England.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

SEARCHES IN THE LIBRARY.

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many such searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling, compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

Copies of all lists of references are filed, so that in many cases it is only necessary to make a typewritten copy, which reduces the cost of searches to a minimum.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

ACCESSIONS TO THE LIBRARY.

From January 8th to February 13th, 1906.

DONATIONS.*

PRACTICAL CEMENT TESTING.

By W. Purves Taylor, Jun. Am. Soc. C. E. Cloth, 9 x 6, illus., 9 + 320 pp. New York, The Myron C. Clark Publishing Co., 1906. $3 net.

It is stated in the preface that the author aims to give a complete description of the methods of handling practical tests of cement. This volume has been designed primarily for the use of the student, the novice, and the practical operator in conducting actual routine tests of cement to determine its suitability for purposes of construction; but it is hoped that both the expert and the engineer who directs this work may also find something of interest in its pages. The general scope includes a description of the properties of cement, the object of the various tests, the methods of conducting them, the common influences and errors most likely to affect the determinations, and the practical interpretation of the results finally obtained. No attempt has been made to consider the practical use of cement and concrete, except in so far as the conditions of actual work regulate the use of the various tests, while the data given are also applicable more to the conduct of tests than to the final use of the material. In other words, the book is intended to cover only the methods and the application of the tests of cement commonly used in routine work, and not to consider theoretical properties, investigations of a research character, nor the use of cement. There is a bibliography and an index of three-and-a-half pages.

Gifts have also been received from the following:

Am. Mathematical Soc. 1 pam.
Arango, R. M. 1 pam.
Atlantic City, N. J.-Board of Water Commrs. 1 pam.
Boston-Transit Comm. 1 bound vol.
Brit. Fire Prevention Committee. 1 pam.
Brooklyn, N. Y.-Commr. of Public Works. 1 bound vol.
Brooks, Frederick. 205 pam.
Brown Hoisting Machinery Co. 1 bound vol.
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Colo. Springs, Colo.-Health Dept. 1 pam.
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Deutsch-Am. Tech. Verbandes. 1 pam.
Dist. of Columbia-Street Cleaning Dept. 1 pam.
Eng. Soc. of Japan. 1 pam.
Fish, J. C. L. 1 pam.
Godfrey, Edward. 1 bound vol.
Great Britain-Under Secy. of State for India. 28 bound vol., 16 vol., 11 pam.
Hague, C. A. 2 pam.
Harrod, B. M. 6 pam. 1 vol.
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Inst. of Civ. Engrs. 1 bound vol.
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Mobile, Jackson & Kansas City R. R. Co. 1 pam.
National Arts Club. 1 pam.
National Board of Fire Underwriters, Committee of Twenty. 2 vol.
New England Cotton Mfrs. Assoc. 1 bound vol.
New South Wales-Govt. Statistician. 3 pam.
New York City-Board of Health. 1 pam.
New York City-Dept. of Docks and Ferries. 1 bound vol.
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Norway-Inspector of Mines. 1 pam.
Ontario, Canada-Provincial Board of Health. 1 pam.
Pennsylvania R. R. System. 1 bound vol.
Platt, T. C. 2 vol.
Ry. Signal Assoc. 1 vol.
Richmond, Fredericksburg & Potomac R. R. Co. 1 pam.
Rochester, N. Y.-Dept. of Public Works. 1 vol.
Royal Philosophical Soc. of Glasgow. 1 vol.
Sachs, Ing.-und Arch. Vereins. 2 vol.
Smithsonian Inst. 1 vol.
South Eastern Ry. Co. 2 pam.

* Unless otherwise specified, books in this list have been donated by the publishers.

Texas-R. R. Comm. 1 vol.
Thompson, R. A. 27 pam.
Transvaal-Irrig. Comm. 1 vol.
Twelvetrees, W. N. 1 pam.
U. S. Bureau of Forestry. 1 pam.
U. S. Bureau of Labor. 1 vol.
U. S. Bureau of Statistics. 1 pam.
U. S. Bureau of Steam Eng. 1 pam.
U. S. Bureau of the Census. 1 bound vol.
U. S. Coast and Geodetic Survey. 1 bound vol., 1 pam., 1 chart.
U. S. Engr. School. 1 pam.
U. S. Geol. Surv. 35 vol., 1 bound vol.
U. S. Interstate Commerce Comm. 18 pam.
U. S. Lake Survey Office. 6 charts.
U. S. Naval Observatory. 1 vol.
U. S. Ordnance Dept. 1 bound vol.
U. S. War Dept. 14 specif.
U. S. Weather Bureau. 1 bound vol.
Univ. of Ill. Agri. Exper. Station. 1 pam.
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Vulcanite Portland Cement Co. 1 pam.
Washington, D. C.-Library of Congress. 1 bound vol.
Western Australia-Dept. of Mines. 1 vol.
Western Australia-Geol. Surv. 4 pam.
Whipple, C. C. 1 pam.
Willson, F. N. 1 bound vol.
Woolson, I. H. 1 pam.
Worcester Polytechnic Inst. 1 vol.

BY PURCHASE.

The Mechanical Engineering of Collieries, Vol. II. By T. Campbell Futers. London, The Chichester Press, 1905.

Sea-Coast Erosion and Remedial Works. By R. G. Allanson-Winn. London, St. Bride's Press, Ltd.

Steam Boilers, Their History and Development. By H. H. P. Powles. London, Archibald Constable & Co., Ltd., 1905.

Architect, Owner and Builder Before the Law. By T. M. Clark. New York, The Macmillan Company; London, Macmillan & Co., Ltd., 1905.

Theory of Structures and Strength of Materials. By Henry T. Bovey. Fourth Edition. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1905.

Outlines of Practical Hygiene. By C. Gilman Currier, M.D., Assoc. Am. Soc. C. E. New York, E. B. Treat & Co., 1905.

Economic Geology of the United States. By Heinrich Ries. New York, The Macmillan Company; London, Macmillan & Co., Ltd., 1905.

Building Construction and Superintendence, Parts 1–2. By F. E. Kidder. New York, William T. Comstock, 1905.

SUMMARY OF ACCESSIONS.

January 8th to February 13th, 1906.

Donations (including 33 duplicates)	458
By purchase	9
Total	467

MEMBERSHIP.

ADDITIONS.

MEMBERS.

		Date of Membership.
ALLIN, THOMAS DAVID. 101 E. Walnut St., Pasadena, Cal.		Jan. 3, 1906
BRENNEKE, WILLIAM GEORGE. (Brenneke & Fay, Cons. Civ. Engrs.), 1009 Fullerton Bldg., St. Louis, Mo.	Assoc. M. M.	Mar. 5, 1902 Jan. 2, 1906
CRIDER, JAMES LELAND. Div. Engr., N. Y., Westchester & Boston Ry., 220 S. Fulton Ave., Mt. Vernon, N. Y.		Jan. 3, 1906
CUSHMAN, WILLIAM HERBERT. Chf. Engr., Juniata Hydro-Elec. Co., Huntingdon, Pa.		Jan. 3, 1906
DUFFIES, EDWARD JOHN. Asst. U. S. Engr., Harbor Beach, Mich.		Jan. 3, 1906
FAY, EDWARD BAYRD. (Brenneke & Fay, Cons. Engrs.), 1009 Fullerton Bldg., St. Louis, Mo.	Assoc. M. M.	Jan. 8, 1902 Jan. 2, 1906
FREELAND, CHESTER SHEPARD. Care, Wm. Hood, 1340 Merchants Exchange Bldg., San Francisco, Cal.		Jan. 3, 1906
LUPFER, EDWARD PAYSON. Asst. Chf. Engr., Buffalo & Susquehanna Ry., 990 Ellicott Sq., Buffalo, N. Y.		Jan. 3, 1906
NICHOLSON, FRANK LEE. 205 N. Park Ave., Norfolk, Va.		Dec. 6, 1905
POLLEYS, WILLIAM VAUGHAN. Res. Engr., Westinghouse, Church, Kerr & Co., Thompson House, East Liverpool, Ohio.		Jan. 3, 1960

ASSOCIATE MEMBERS.

ADAMS, EDWARD MAGUIRE. 1st Lieut., Adjutant, 3d Battalion of Engrs., Fort Leavenworth, Kans.	Jun. Assoc. M.	Oct. 7, 1902 Jan. 3, 1906
BELL, THOMAS K. Chf. Engr., Wm. Wharton, Jr. & Co., Inc., 25th and Washington Ave., Philadelphia, Pa.		Jan. 3, 1906
BLACKWELL, PAUL ALEXANDER. Canonsburg, Pa.	Jun. Assoc. M.	Dec. 4, 1900 Jan. 3, 1906
COCHRANE, VICTOR HUGO. 55 Water St., Pittsburg, Pa.		Oct. 4, 1905
DEGRAFF, HARRY WESTBROOK. Asst. Engr. in Chg., Res. No. 3, N. Y. State Barge Canal, Fonda, N. Y.		Jan. 3, 1906
DEMPSTER, OSBORNE JOEL. Herkimer, N. Y.		Jan. 3, 1906
DILKS, LORENZO CARLISLE. Contr. Mgr., The Eastern Steel Co., 71 Broadway, New York City.		Jan. 3, 1906
HARSHBARGER, ELMER DWIGHT. First Asst. Engr., Bureau of Filtration, Dept. of Public Works, Pittsburg, Pa.	Jun. Assoc. M.	Nov. 5, 1901 Jan. 3, 1906
HERMANNS, FRANK EDWARD. With J. G. White & Co., 43 Exchange Pl., New York City.		Jan. 3, 1906

ASSOCIATE MEMBERS (*Continued*).

	Date of Membership.
PETERSON, JOHN. Chf. Draftsman, Dept. of Water Supply, Gas and Electricity, Borough of Brooklyn, Room 41, Municipal Bldg., Brooklyn, N. Y.	Jan. 3, 1906
SARGENT, ARTHUR WINTHROP. U. S. Engr. Office, Seattle, Wash.	Sept. 6, 1905
SHAW, ARTHUR MONROE. Cons. and Contr. Engr., Dixon, Ill.	Jan. 3, 1906
SHERMAN, EDWARD CLAYTON. Asst. Engr., Charles River Basin Comm., 367 Boylston St., Boston, Mass.	Jan. 3, 1906
STODDARD, RAYMOND FRENCH. Asst. Engr., N. Y., N. H. &. H. R. R. Co., Milford, Conn.	Jan. 3, 1906
THOMSON, SAMUEL FORSYTHE. Asst. Engr., Rapid Transit R. R. Comm., 4 Court Sq., Brooklyn, N. Y.	Jan. 3, 1906
WAIT, OWEN ADELBERT. Designing Engr., Great Northern Power Co., 316 Providence Bldg., Duluth, Minn.	Dec. 6, 1905

ASSOCIATES.

BRUCE, JOHN MOFFATT. 42 Broadway, New York City.	Jan. 3, 1906
CONARD, WILLIAM ROBERTS. Burlington, N. J.	Jan. 3, 1906

JUNIORS.

ABRAMS, DUFF ANDREW. 703 South 3d St., Champaign, Ill.	Jan. 2, 1906
BLACK, EDWARD FRYLING. 323 Baronne St., New Orleans, La.	Oct. 31, 1905
COLLINS, FRANCIS WINFIELD. 535 West 142d St., New York City.	Jan. 2, 1906
ESTEN, HOWARD FOSS. 102 Cedar St., Pawtucket, R. I.	Jan. 2, 1906
FEDERLEIN, WALTER GOTTLIEB. Asst. Engr. in Chg. of Sewers, Hudson Companies, 111 Broadway, New York City.	Jan. 2, 1906
HIGGINS, CHARLES HOUCHIN. Engr. for F. M. Stillman Co., 26 Exchange Pl., Jersey City, N. J.	Jan. 2, 1906
HITT, RODNEY. Associate Editor, *Railroad Gazette*, 83 Fulton St., New York City.	Sept. 5, 1905
HOVEY, RAY PALMER. With the Am. Bridge Co., P. O. Box 459, Ambridge, Pa.	Jan. 2, 1906
HUNTSMAN, FRANK C. Macon, Mo.	Sept. 5, 1905
LEE, CHARLES HAMILTON. 1108 Union Trust Bldg., Los Angeles, Cal.	Jan. 2, 1906
LOUGHRAN, HAROLD SCOTT. Asst. Engr., Dept. of Sewers, City Hall, New Rochelle, N. Y.	Jan. 2, 1906
MAUPIN, EDGAR STAPLES. U. S. Govt. Fleet, Greenville, Miss.	Sept. 5, 1905

JUNIORS (*Continued*).

	Date of Membership.
NICHOL, HENRY SCHELL. With Milliken Bros., 11 Broadway, New York City	Jan. 2, 1906
PENDLETON, DAVID ELLIOTT. Box 404, Vicksburg, Miss.	Jan. 2, 1906
POWELL, WILLIAM JENNER. U. S. Navy Barge No. 122, Care, Postmaster, Mobile, Ala	Jan. 2, 1906
SHEMA, JOSEPH. 2239 Essex St., Baltimore, Md	Oct. 3, 1905
WYMAN, ALFRED MARSHALL. Asst. Engr., Rapid Transit Comm., 203 North 16th St., E. Orange, N. J	Jan. 2, 1906

RESIGNATIONS.

MEMBERS.

	Date of Resignation.
GREENE, BENJAMIN DWIGHT	December 31, 1905

DEATHS.

BOTH, CARL CHRISTIAN ADOLPH. Elected Member, September 2d, 1891; died January 12th, 1906.

BOWMAN, JOSEPH HOCKMAN. Elected Associate Member, December 2d, 1903; date of death unknown.

GIBSON, WILLIAM, JR. Elected Associate, September 5th, 1888; died April 19th, 1905.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(January 7th to February 9th, 1906.)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Street Railway Journal*, New York City. Issues for first Saturday of each month 20c., other issues 10c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England, 25c.
(22) *Iron and Coal Trades Review*, London, England, 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Engineer*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, $1.
(29) *Journal*, Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l'Assoc. des Ing. Sortis des Ecoles Speciales de Gand*, Brussels, Belgium.
(32) *Memoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Genie Civil*, Paris, France.
(34) *Portefouille Economique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Review de Mecanique*, Paris, France.
(38) *Revue Generale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill., 10c.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Proceedings*, Am. Inst. Elect. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussees*, Paris, France.
(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 8c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.

(54) *Transactions*, Am. Soc. C. E., New York City, $5.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $5.
(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Eng. Soc. W. Pa., 410 Penn Ave., Pittsburg, Pa., 50c.
(59) *Transactions*, Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 20c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 15c.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England.
(70) *Engineering Review*, New York City, 10c.
(71) *Journal*, Iron and Steel Inst., London, England.
(72) *Street Railway Review*, Chicago, 30c.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England.
(78) *Beton und Eisen*, Vienna, Austria.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie-Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(82) *Dinglers Polytechnisches Journal*, Berlin, Germany.
(83) *Progressive Age*, New York City, 15c.

LIST OF ARTICLES.

Bridge.

The Ferry Bridge across the Ship Canal at Duluth, Minnesota.* C. A. P. Turner, M. Am. Soc. C. E. (54) Vol. 55.
A Few Points in the Design of Reinforced Concrete Arches. B. R. Leffler, Assoc. M. Am. Soc. C. E. (54) Vol. 55.
A Rational Form of Stiffened Suspension Bridge.* Gustav Lindenthal, M. Am. Soc. C. E. (54) Vol. 55.
Theory and Formulas for the Analytical Computation of a Three-Span Suspension Bridge with Braced Cable.* Leon S. Moisseiff, Assoc. M. Am. Soc. C. E. (54) Vol. 55.
A New Swing Bridge at Copenhagen, Denmark.* H. C. V. Moeller. (54) Vol. 55.
The Reconstruction of the Baltimore and Ohio Railroad Bridge over the Ohio River, at Benwood, West Virginia.* J. E. Greiner, M. Am. Soc. C. E. (54) Vol. 55.
The Strengthening and Maintenance of Early Iron Bridges.* William Marriott, M. Inst. C. E. (63) Vol. 162.
Some further Tests of Reinforced Concrete Beams; C., M. & St. P. Ry. J. J. Harding. (4) Dec.
Some Notable American Railway Bridges.* James G. Walton. (10) Jan.
The Lower Chords of the Island Span of the Blackwell's Island Bridge.* (14) Jan. 6.
The Design of High Abutments. Wm. M. Torrance. (13) Jan. 11.
Main Vertical and Inclined Posts, Island Span, Blackwell's Island Bridge.* (14) Jan. 27.
Substructure of Potomac River Highway Bridge, Washington, D. C.* (14) Jan. 27.
Ueber die Berechnung von Schiffbrücken mit Gelenken.* H. Müller-Breslau. (49) Pts. 1-3, 1906.
Amerikanische Klappbrücken.* Georg v. Hanffstengel. (82) Serial beginning Jan. 6.

Electrical.

Unsolved Problems in Electrical Engineering. Rookes Evelyn Bell Crompton, M. Inst. C. E. (63) Vol. 162.
A Study of a Single-Phase Series Motor.* George I. Rhodes. (7) Dec.
A Note on the Calculation of the Armature Reaction of Alternators.* Waldo V. Lyon. (7) Dec.
The Measurement of High Frequency Currents and Electric Waves. J. A. Fleming. (29) Serial beginning Dec. 29.
Electric Mains for Power Transmission Work. John T. Morris. (Abstract of Paper read before the Junior Inst. of Engrs.) (73) Dec. 29.

*Illustrated.

Electrical—(Continued).

Power Plant Economics. Henry G. Stott. (42) Jan.
A Self-Exciting Alternator.* E. F. Alexanderson. (42) Jan.
Notes on Heavy Electric Switch Gear. J. Whitcher. (Abstract of Paper read before the Rugby Eng. Soc.) (73) Jan. 5.
The "Kaiserwerke" Electricity Works in the Tyrol.* (73) Jan. 5.
Wave Shapes in Three-Phase Transformers.* R. C. Clinker. (73) Jan. 5.
The Electrical Equipment at Messrs. J. Hopkinson & Co.'s Works, Huddersfield.* (26) Jan. 5.
Electric Lighting at Buncrana, Co. Donegal.* (26) Jan. 5.
Power Generation and Distribution on the System of the Public Service Corporation of New Jersey.* (17) Serial beginning Jan. 6.
Turbo Alternator for Glasgow.* (17) Jan. 6.
The Marion (Hackensack River) Station of the Public Service Corporation of New Jersey.* (27) Jan. 6.
The Architecture of Continental Power Plants.* (27) Jan. 6.
Exposed Circuit Wiring.* Louis J. Auerbacher. (27) Jan. 6.
District Supply in Rural Communities.* (27) Jan. 6.
Central Station Economics in Massachusetts: A Study of Two Typical Medium-Sized Companies. (27) Jan. 6.
Core Type Transformers for High Tension Power Transmission.* A. H. Pikler. (27) Jan. 6.
North Mountain Power Company's Hydro-Electric Plant.* (27) Jan. 6; (14) Jan. 6.
The London-Glasgow Underground Telegraph System.* (26) Serial beginning Jan. 12.
The London-Glasgow Telegraph Cable.* (73) Jan. 12.
Notes on a Wireless Telegraph Station.* C. C. F. Monckton. (73) Jan. 12.
Design of Turbo-Alternators.* H. S. Meyer. (73) Jan. 12.
Specifications for Line Wire. F. F. Fowle. (Paper read before the Ry. Signal Assoc.) (40) Jan. 12.
Hydro-Electric Installation at Sewalls Falls, N. H.* (18) Jan. 13.
The Electrical Distribution System of the Public Service Corporation of New Jersey.* G. U. G. Holman. (27) Serial beginning Jan. 13.
The Marion Power Station of the Public Service Corporation of New Jersey.* (14) Jan. 13.
The Post Office Telephone System: The "City" Exchange.* (26) Serial beginning Jan. 26.
Electric Power Distribution in North Wales.* (73) Serial beginning Jan. 26.
The New Post Office City Telephone Exchange.* (73) Jan. 26.
A Modern Central-Station Plant.* (64) Feb.
Lightning Protection.* J. V. E. Titus. (Paper read before the Ohio Interurban Ry. Assoc.) (17) Feb. 3.
The Springfield, Ill., Light, Heat and Power Co.'s Station and System.* (27) Feb. 3.
The DeForest Syntonic System of Wireless Telegraphy. A. Frederick Collins. (19) Feb. 10.
Zur Theorie der Wechselstromkreise. Leo Lichtenstein. (82) Serial beginning Jan. 20.
Ueber Hochspannungsisolatoren. J. Pusch. (80) Jan. 30.

Marine.

Shipbuilding for the Navy. Lord Brassey, Assoc. Inst. C. E. (63) Vol. 162.
The Cruiser.* William Hovgaard. (Read before the Soc. of Naval Archts. and Marine Engrs.) (7) Dec.
Submarine Signaling.* Henry R. Gilson. (7) Dec.
The Largest Turbine Steamship in the World: The *Carmania* of the Cunard Line.* Archibald S. Hurd. (10) Jan.
A Shipbuilding Cableway.* (12) Jan. 19.
Torsion Indicator Diagrams of Marine Engines.* (11) Jan. 26.
Le Paquebot à Turbines *Carmania* de la Compagnie Cunard.* L. Piaud. (33) Jan. 16.
Der Transatlantische Turbinendampfer *Carmania.** W. Kaemmerer. (48) Jan. 6.
Dockanlage für Torpedoboote auf der Kaiserlichen Werft Kiel.* Ph. von Klitzing. (48) Jan. 20.

Mechanical.

Preliminary Report of the Committee Appointed on the 6th of November, 1903, to Consider and Report to the Council on the Standards of Efficiency of Internal Combustion Engines.* (63) Vol. 162.
A Portable Apparatus for the Analysis of Flue Gases. Charles Joseph Wilson. (63) Vol. 162.
The Strength of Shafts Subject to Small Forces Rhythmically Applied. Charles Chree, Henry Riall Sankey, M. Inst. C. E., and William Ernest Wyatt Millington, Stud. Inst. C. E. (63) Vol. 162.

* Illustrated.

Mechanical—(Continued).

Advantages and Applications of the Electric Drive. F. B. Crocker and M. Arendt. (6) Nov.
The Oxygen Blow-Pipe in Iron and Steel Welding.* (22) Dec. 29.
Tests of De Laval Steam Turbine.* Thomas B. Morley. (11) Dec. 29.
Standard Alloys. John F. Buchanan. (From *The Foundry*.) (47) Dec. 30.
Testing of High-Power Modern Gas Engines.* William H. Spiller. (41) Jan.
The Conditions of Fan-Blower Design. Walter B. Snow. (10) Jan.
Power in Tall Office Buildings: Its Cost and Distribution.* Charles H. Benjamin. (10) Jan.
Heat Insulation: Its Principles as Related to Cold Storage Practice. J. B. d'Homergue. (58) Jan.
A New Machine for Bending Tests.* E. Probst. (67) Jan.; (62) Feb. 1.
The Action of Slightly Alkaline Waters on Iron. Cecil H. Cribb. (Abstract of Paper read before the Soc. of Public Analysts.) (11) Jan. 5.
Note on Steam Turbines.* H. Riall Sankey, M. Inst. C. E. (11) Jan. 5.
The Oechelhauser Gas-Engine.* (11) Serial beginning Jan. 5.
Modern Gas Engine Power Plants.* Franz Koester. (14) Jan. 6.
The New Works of the Milwaukee Gas Light Co.* (13) Jan. 11.
The "Ados" CO_2 Recorder.* (11) Jan. 12.
The Guillery Hardness-Testing Apparatus.* (11) Jan. 12.
Methods of "Changing Speed" in Electric Motor Driving.* E. Kilburn Scott. (22) Jan. 12.
Cast Iron (in the foundry). Herbert Pilkington. (22) Jan. 12.
Fireclays and Moulding Sands. Percy Longmuir. (Paper read before the British Foundrymen's Assoc.) (22) Jan. 12.
A New Machine for Bending Tests. (14) Jan. 13.
Vertical Retorts for the Production of Illuminating Gas.* W. R. Herring, M. Inst. C. E. (66) Jan. 16.
Shear Stress and Permanent Angular Strain (in turning tools).* W. C. Popplewell, A. M. Inst. C. E. (12) Jan. 19.
A Rational Method of Cooling Gas Engine Cylinders.* S. M. Howell. (46) Jan. 20.
Power Required to Thread, Twist and Split Wrought Iron and Mild Steel Pipe.* T. N. Thomson. (Abstract of Paper read before the Amer. Soc. of Heat. and Vent. Engrs.) (20) Jan. 25; (70) Jan.; (24) Jan. 29.
Some Notes on Modern German Rolling Mills.* (22) Serial beginning Jan. 26.
Superheated Steam.* Michael Longridge, M. Inst. C. E. (Lecture delivered at the Bradford Tech. School.) (47) Serial beginning Jan. 27.
Smoke Abatement: A Report on the London Smoke Abatement Conference: December 13-15, 1905. John B. C. Kershaw. (10) Feb.
Trials of Suction Gas-Producer Plants. (64) Feb.
Applications of Pneumatic Power in the Machine Shop. R. Emerson. (9) Feb.
High Power Gas Engine Electric Plants.* Frank C. Perkins. (41) Feb.
Nickel Steel and Its Application to Boiler Construction. G. B. Waterhouse. (20) Feb. 8.
The Helicoptere: Santos-Dumont's Latest Flying Machine.* L. Ramakers. (46) Feb. 10.
Dampfturbinen mit Geschwindigkeitsstufen und mit Druckstufen. Fritz Krull. (53) Dec. 29.
Die Edison-Portlandzement-Fabrik in Newvillage, New Jersey, V. St. A.* (80) Dec. 30.
Die Bildung von Rissen in Kesselblechen.* C. Bach. (48) Jan. 6.
Die Autogene Schweissung der Metalle. E. Wiss. (48) Jan. 13.
Schwimmender Kohlenspeicher für 12000 t.* W. Kaemmerer. (48) Jan. 27.

Metallurgical.

The Effect of Variations in the Speed of Crushing Machinery upon the Production of Undersized Material. H. W. Gartrell. (6) Nov.
Steel Castings and the Constitution of Steel.* Percy Longmuir. (From *Iron Trade Review*.) (22) Dec. 29.
Determination of Carbon in Steel by Direct Ignition with Red Lead. Charles Morris Johnson. (58) Jan.
Die Herstellung von Eisen und Stahl auf Elektrischem Wege.* Albert Neuburger. (52) Serial beginning Nov. 15.
Die Brikettierung der Eisenerze und die Prüfung der Erzziegel. H. Wedding. (50) Serial beginning Jan. 1.
Einiges aus der Metallographischen Praxis.* E. Heyn. (Paper read before The Deutscher Verband für die Materialprüfungen der Technik.) (50) Jan. 1.
Ueber den Gegenwärtigen Stand der Gichtgasreinigung. Meyjes. (Paper read before the Südwestdeutsch-Luxemburgische Eisenhütte.) (50) Jan. 1.

Mining.

A Hydraulic-Pneumatic Mine Door Opener.* L. L. Logan. (45) Dec. 29.
Some Electric Installations in European Mines.* Emile Guarini. (45) Dec. 29.

* Illustrated.

Mining—(Continued).

A Concrete Breaker.* (45) Jan.
Fuel Economy at Bituminous Coal Mines in Pennsylvania. C. E. Watts. (58) Jan.
Shaft Sinking in Quicksand.* Geo. C. McFarlane. (16) Jan. 20.
Track Construction in Mines. Leo Gluck. (Abstract of Paper read before the Illinois Soc. of Engrs. and Surveyors.) (13) Jan. 25.
The Manufacture, Application and Distribution of Electric Cables for Collieries.* George G. L. Preece. (Paper read before the Nat. Assoc. of Colliery Mgrs.) (22) Jan. 26.
Tailing Disposal by Gold Dredges.* J. P. Hutchins. (16) Feb. 3.

Miscellaneous.

The Estimation of Costs. A. W. Farnsworth. (From Paper read before the Coventry Eng. Soc.) (14) Feb. 3.

Municipal.

County Road Construction: A Topical Discussion. (58) Jan.
Municipal Ownership (electric lighting plant). L. A. Redman. (83) Feb. 1.
The Construction of Pavements in Chicago. J. A. Moore. (From Paper read before the Ill. Soc. of Engrs. and Survs.) (14) Feb. 3.

Railroad.

Round-House Framing.* R. D. Coombs, Assoc. M. Am. Soc. C. E. (54) Vol. 55.
The Single-Phase Railway System. Charles F. Scott. (Paper prepared for Amer. St. Ry. Assoc.) (47) Dec. 30.
The Simplon Tunnel. (41) Jan; (45) Jan.
Contractors' Locomotives.* J. F. Gairns. (10) Jan.
High-Speed Electric Railroads. Henry G. Morris. (2) Jan.
Throwing Devices for Tongue Switches.* T. A. Gerlach. (72) Jan.
Extension and Improvements of the Chicago & Milwaukee Electric Railroad Co.* (72) Jan.
The New Shops of the Portland Railroad Co. (72) Jan.
Three-Cylinder Balanced Compound "Atlantic Engines;" Great Central Railway.* (21) Jan.
Reconstruction of Haydon Square Goods Depot, London and North-Western Railway.* Chas. S. Lake. (21) Jan.
Coach Painting in India. J. Wilson Hall. (21) Jan.
Electrification of the Paris-Orleans Suburban Line.* (11) Jan. 5.
20,000 Volt Single-Phase Locomotive for Sweden.* (73) Jan. 5.
Six-Coupled Engines on the Glasgow and South-Western Railway.* (12) Jan. 5.
The New Hoboken Freight Terminal of the Lackawanna R. R.* (14) Jan. 6.
Concrete Retaining Walls at the New York Central Terminal, New York. (14) Jan. 6.
Sedalia Shops, Missouri Pacific Ry.* (18) Jan. 6.
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An Example of the Legitimate Use of Water for Domestic Purposes. K. F. Cooper, Jun. Am. Soc. C. E. (54) Vol. 55.
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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THE PANAMA CANAL.

By A. G. Menocal, M. Am. Soc. C. E.*

To be Presented April 4th, 1906.

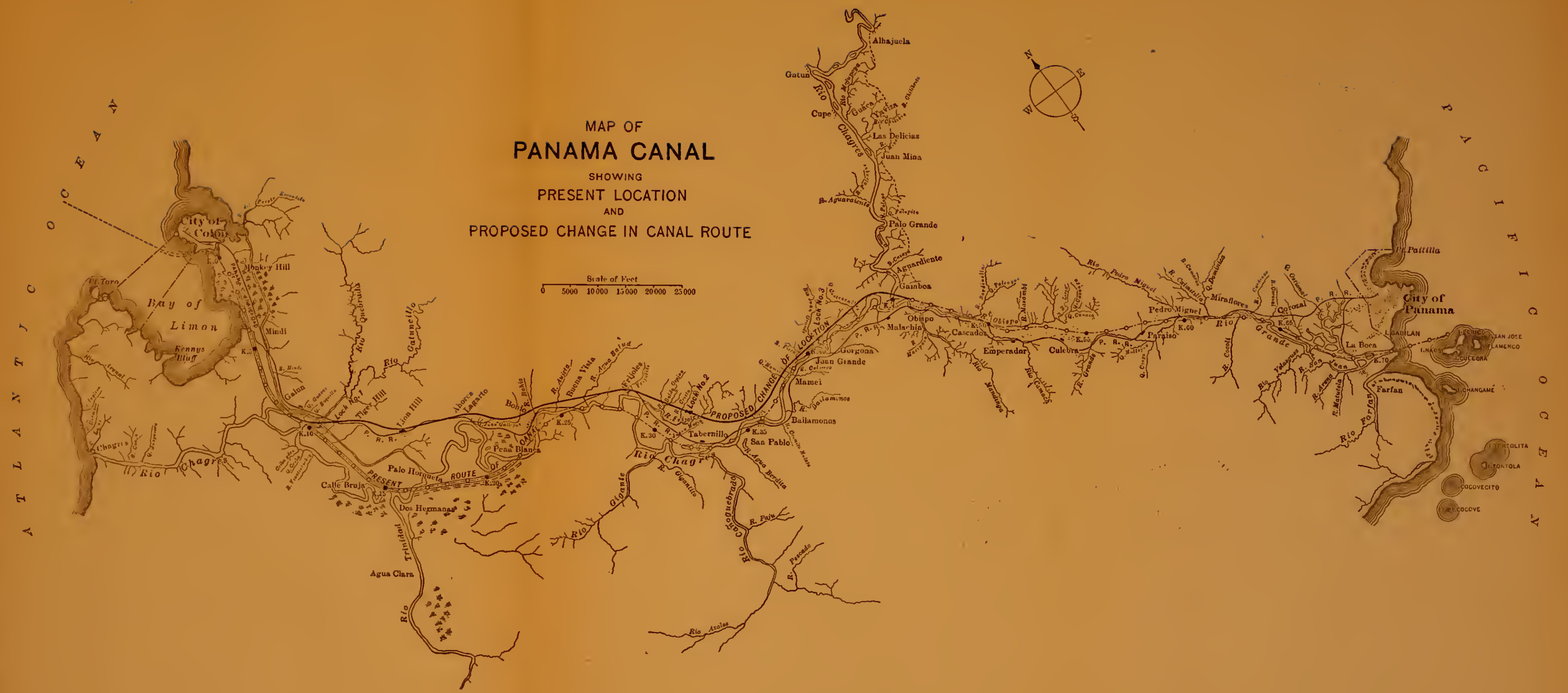
The most difficult engineering problem involved in the construction of a canal at Panama is the control of River Chagres. It enters as an important factor in the design of either a lock or a sea-level canal, and the divergence of opinions, among engineers called upon to decide as to the type of canal best adapted to meet the physical conditions prevailing in the Isthmus, can be traced to the difficulties connected with that river, both as regards the control of the floods, in all types of canal, and the provisions for an ample water supply for operating it during the dry season, in the case of a canal with locks.

It is evident that a lock canal is the most economical type, both in cost and time of construction, and that the sea-level proposition is born either of sentiment or of a belief that by its adoption the difficulties connected with the river can best be overcome. It is well known that a sea-level canal pertaining to the nature of a strait is not possible at Panama. The tidal fluctuation of 20 ft. at the Pacific terminus while the Atlantic end is practically tide-

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* Civil Engineer, U. S. N., *Retired*.

PLATE I.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1906.
MENOCAL ON
THE PANAMA CANAL.



less, makes imperative the introduction of a tide lock at Panama, by which ships can be locked up or down, into or from the canal, depending on the stage of the sea level at the time of taking or leaving the waterway. That tidal lock will limit the number of vessels passing through the canal just as much as a series of locks in a lock canal. Some time will be spent in passing each additional lock introduced, and this should not exceed 30 minutes at each lockage, so that the additional time consumed in passing through a canal with six locks would not be more than 3 hours; an insignificant delay for a ship which has saved thousands of miles by taking the canal route.

Considering, therefore, the cost and time saved in constructing a lock canal, as compared with one at sea level, as well as the elimination of the uncertainties and engineering difficulties connected with the latter, it seems that the former type should be adopted, provided the River Chagres can be effectually controlled, an ample water supply provided, and difficult engineering problems avoided.

The object of this paper is to submit for discussion by the Society a modification of the canal route recommended by the Isthmian Canal Commission of 1899-1901 for a lock canal, by which the River Chagres may be kept under absolute control, its channel being left free to carry off the floods, an abundant supply of water close at hand secured for the operations of the canal, all doubtful problems eliminated, without increasing the cost estimated by the Commission, and with a saving of time in the execution of the work. The change in location recommended begins at Kilometer 46, just south of where the present location meets the Chagres River. On a prolongation of the tangent ending at this point it crosses the River Chagres in the vicinity of Gamboa, and thence, upon the line located by the United States Surveying Expedition of 1875, the canal is kept north of the river, meeting the present location again at Kilometer 9 and coinciding with it to the Harbor of Colon. (Plate I.)

The crossing of the Chagres is proposed to be accomplished by means of a combined dam, viaduct, and controlling works. The dam impounds the river at a maximum elevation of 111 ft. above mean sea level and a minimum of 106 ft. at the end of the dry season.

The canal crosses the river in a viaduct, which is also part of the dam, with a summit elevation of 96 ft., the bottom width being 150 ft., and the depth of water 35 ft. As the minimum elevation of the lake formed by the dam is estimated to be at no time lower than 106, the canal can be supplied with the water needed for its operation by short pipes passing through the dam and discharging under the water surface. The water supply is thus kept under perfect control, to be drawn as needed, and the canal can be maintained at a uniform level.

The river controlling works consist of seven compartments or gate-wells (see Plates III and IV) and twenty-one 10-ft. metal-lined sluices passing through the structure. These sluices are provided with gates, three in each compartment, operated from the upper platform, by which the river flow can be completely shut off, which may be desirable in the dry season when the river flow may, at times, be less than that required for operating the canal; or the gates may be partially or entirely opened to allow the free passage of floods. With the water in the lake at an elevation of 111 and the lower river in flood, 30 ft. above low water at the viaduct, the twenty-one sluices, under such extreme conditions, will be capable of passing 78 000 cu. ft. per sec., which is far in excess of any known flood discharge at Gamboa. Gate-wells are provided with sill walls rising 10 ft. above low water in the river, intended to arrest any heavy silt or stones rolling on the river bottom. Above the sill walls, the intakes are provided with grooves for the insertion of movable gates, which can be lowered into place or taken up by derricks in the upper platform. With these gates the intakes can be closed and the water drained from a well for examination or repair when needed.

The intakes are protected by heavy iron gratings with openings 2 ft. square to intercept snags, or other floating debris brought down by floods, which might obstruct the sluices or impede the operation of the gates. The water for operating the canal is taken from below the surface of the lake, after passing through the grating, and should be practically free from floating debris. It is discharged into the canal below the surface, so as to cause the least disturbance, or it may be made to discharge through conduits under the bottom of the canal. The upper platform, formed by the dams

PLATE II.
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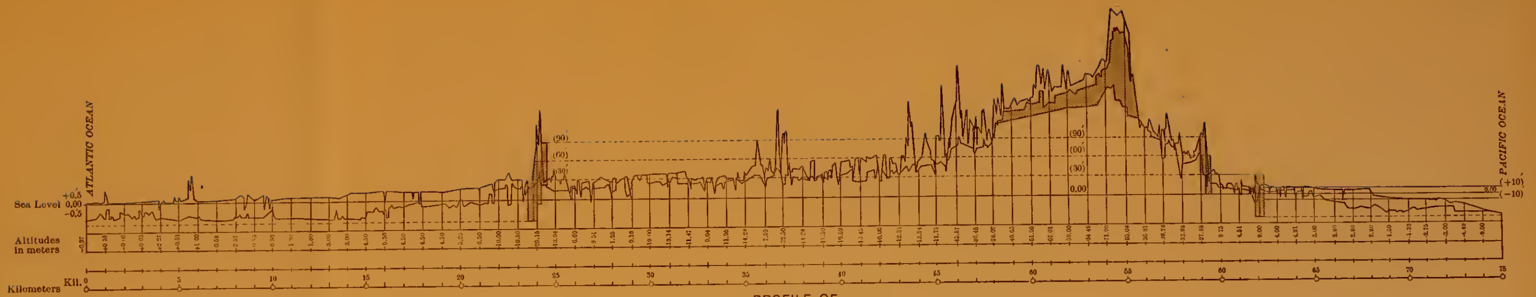
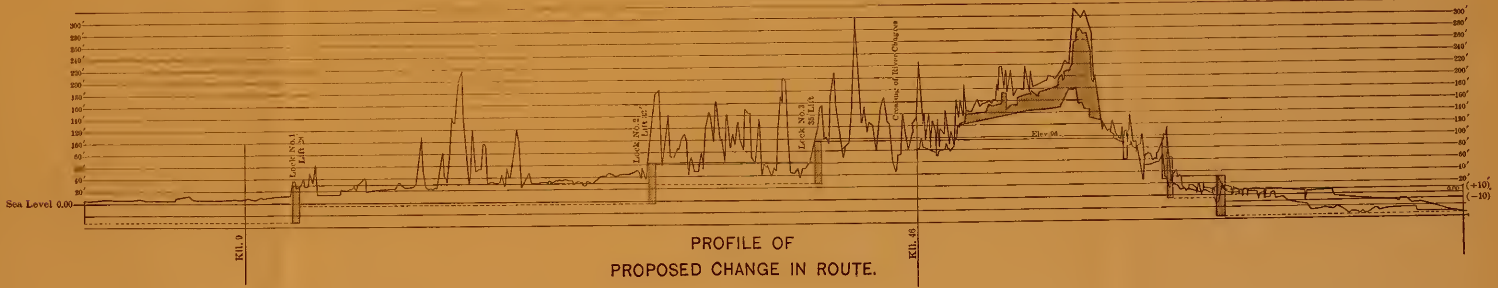


PROFILE OF
PRESENT ROUTE OF CANAL.



PROFILE OF
PROPOSED CHANGE IN ROUTE.

and intake walls, 32 ft. wide, can be roofed for the protection of the gate-operating machinery and the employees manipulating it. The control of the floods and of the water supply can thus be concentrated in one structure.

The combined structure, of reinforced concrete, 232 ft. wide and resting on hard rock, with an elevation of only 118 ft. above the foundations, possesses superabundance of strength and all the conditions of stability and durability essential in a work of this kind. It is believed that it solves effectually the difficult problems connected with the Chagres River, and removes the risks and uncertainties involved in the construction and permanency of dams designed to rest on doubtful foundations. It simplifies the building of the canal, reducing it to a comparatively ordinary engineering work of construction, admitting of an estimate of the cost and time of execution not to be upset by river floods.

The rock surface under the proposed structure has been plotted from data obtained recently by numerous borings made by the engineers of the Isthmian Canal Commission in the vicinity of Gamboa while exploring for the location of a high dam.

The line proposed from Kilometer 9 to Kilometer 46 of the present canal location was surveyed by the United States Government's Surveying Expedition of 1875 under the direction of the writer as Chief Engineer. It is, however, but a trial location which a limited appropriation did not permit to be surveyed in detail. A final location, after careful development of the topography, will, doubtless, bring about important improvements tending to reduce the excavation and the amount of curvature. It is perfectly practical as now laid down, is about a mile shorter than the present canal location between Kilometers 9 and 46, and involves no engineering difficulties of execution.

With good rock foundations and a wide cross-section of the river at the point of crossing, the viaduct-dam can be constructed with the use of coffer-dams in that portion of the work resting on foundations below water level. The summit level of the canal has been placed at Elevation 96, not imposed by the conditions of the problem, but because that is believed to be the most economical elevation. It is evident that, by lowering the sluices, either by reducing their diameter, increasing their number, if necessary, or

by giving them the shape of inverted siphons, the canal upper level can be lowered several feet, but such modification is not regarded favorably. It would reduce somewhat the lift of the upper locks, which, as herein suggested, is moderate, but at an unwarranted additional cost of excavation.

An alternative plan, which has received consideration, consists in bringing the canal into the lake created by the dam and then crossing the river through the lake instead of the viaduct. By such a change of plan, the proposed works would be reduced to the dam and controlling works, and the cost thereby considerably diminished. The writer is of the opinion that such a modification of the scheme is not desirable. The saving in the cost of the structure would probably be more than offset by an increase in excavation; and cross-currents in the lake, fluctuations of level in the canal, and floating objects brought down by river floods would be serious obstacles to navigation, from which the viaduct plan is entirely free.

Without a contour map of the river basin above Gamboa, the writer is unable to ascertain the superficial area of the lake created by the dam. With a surface elevation of 111 ft. at the dam, the lake would extend to beyond Alhajuela, where the low-water level will be raised about 8 ft. Considering the broad expansions of the river basin and the re-entering valleys of its tributaries, it is contended that the lake will have an area large enough to receive the greatest floods, without violent fluctuations of level, while the whole flow is being carried off by the sluices, and will have ample storage capacity for an abundant water supply for operating the canal in the dry season within the range of Elevations 106 and 111. No reservoir is needed for the storage of floods, and the precise elevations within which the water supply can be assured is a matter of detail which does not alter the essential features of the proposed scheme.

Pending a resurvey of the new location proposed between Kilometers 9 and 46 (which will doubtless result in material improvement in alignment and elevation), and the borings necessary for a classification of the material to be removed, it is not possible to obtain a close estimate of cost of the modified plans.

However, taking for comparison the estimated cost of the

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MENOCAL ON
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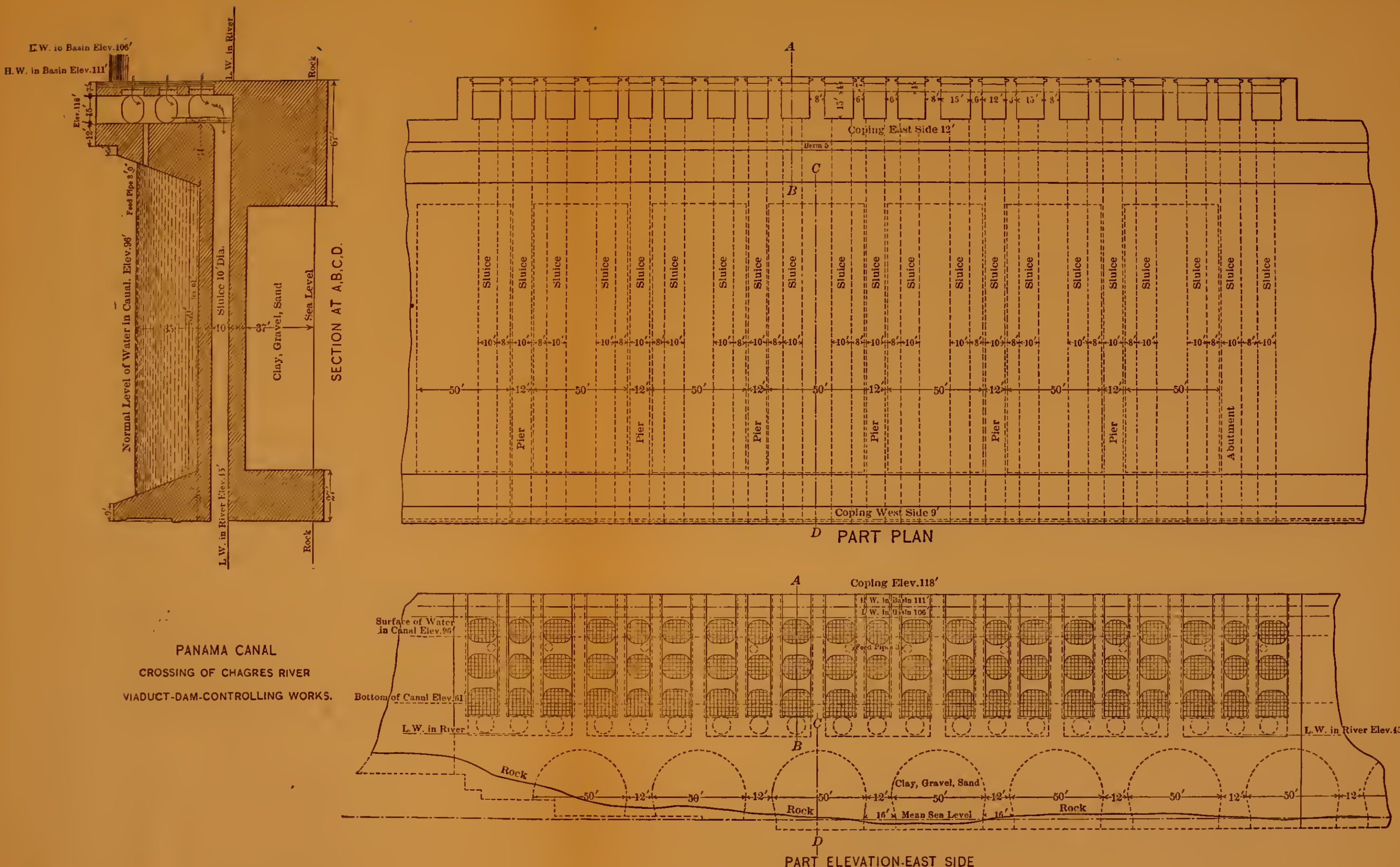


PANAMA CANAL
CROSSING OF CHAGRES RIVER
VIADUCT-DAM-CONTROLLING WORKS.

Isthmian Canal Commission's plan for a lock canal with a fluctuating summit level between a maximum of 92.5 ft. and a minimum of 82 ft. above mean sea level, it is possible to arrive at the conclusion that the cost of the modified plan proposed will not exceed the estimate of the Commission, and may fall considerably below it. In any case, it is claimed that if the time of construction can be lessened, and the permanency of the canal at a reduced cost of maintenance can be assured, this would be worth several millions.

The works of the Chagres crossing are estimated to cost $5 275 000. Upon the new route proposed about 52 000 000 cu. yd. will have to be removed from the Harbor of Colon to Kilometer 46, of which 10 000 000 cu. yd. are contained in the sea-level section from the Harbor to Kilometer 9, where it joins the new line. The latter quantity is probably material which can be dredged, the other 42 000 000 cu. yd. being dry soil with some rock in the high ridges crossed.

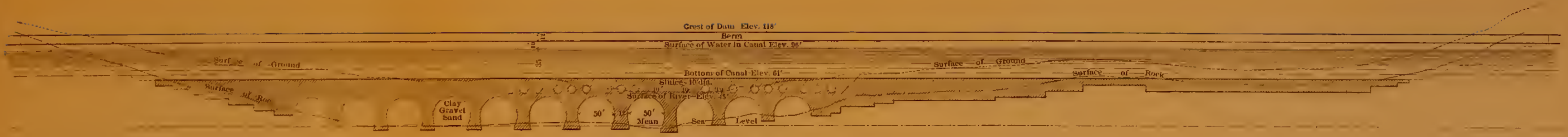
To offset the cost of these works, the following amounts are taken from the estimate of the Commission for work between Colon and Kilometer 45, which would be eliminated by the adoption of the modified plan.

Harbor of Colon to Bohio Locks, including levees	$11 099 839
Lake Bohio	2 952 154
Obispo Gates	295 434
Bohio Dam	6 369 640
Gigante Spillway	2 448 076
Pena Blanca Outlet	1 999 982
Diversion of the Panama Railroad	1 267 500
Total	$26 432 625

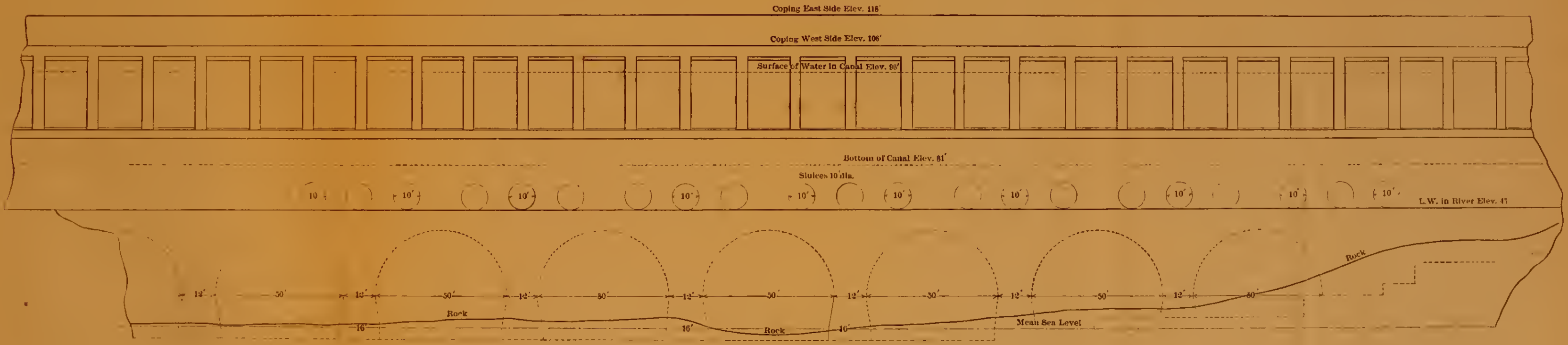
In addition to this total, there would also be a saving of 14 ft. depth in the Culebra Cut, by raising the bottom of the canal from Elevation 47 to Elevation 61, which may be estimated to contain about 20 000 000 cu. yd. The item, $11 567 275, in the Commission's estimate for the two high-lift locks at Bohio will probably balance the cost of the three locks of about the same aggregate lift proposed on the new route now suggested.

As the basin of the Chagres below Gamboa is not to be flooded by the creation of a lake, the diversion of the Panama Railroad becomes unnecessary. The road can cross the canal by draw-bridges, or under it by short tunnels where necessary, and, eventually it can be built on the berm of the canal if so desired.

PLATE IV.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1906.
MENOCAL ON
THE PANAMA CANAL.



LONGITUDINAL SECTION THROUGH AXIS OF CANAL



PART ELEVATION—WEST SIDE

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PAPERS AND DISCUSSIONS.

A COMPLETE ANALYSIS OF GENERAL FLEXURE IN A STRAIGHT BAR OF UNIFORM CROSS-SECTION.

By L. J. Johnson, M. Am. Soc. C. E.

To be Presented April 18th, 1906.

1. *Introduction.*—General flexure is to be understood in this paper to include all cases of stress in a right section in which there is normal stress (tension or compression) in any part of the section. Accordingly, it includes (1) pure flexure; (2) combined flexure, and tension or compression; and (3) pure tension or pure compression. These three are merely particular cases in which the neutral axis is at a zero, an intermediate, or an infinite, distance, respectively, from the center of gravity of the stressed section.

The correlative to general flexure would be general torsion, the latter covering all cases in which tangential stress (shear) is involved, just as the former covers the whole ground of normal stress. Together, they would include all cases of stress. Each may be looked upon as the result of resistance to rotary deformation, the axis of rotation being, in the former case, in the plane of the stressed section, in the latter, normal to that plane.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The term analysis is to be understood to mean the study of the distribution of stress over the stressed section.

The word complete is used in the title in contradistinction to the partial analysis to which almost all writers of English and American textbooks limit themselves. The familiar analysis of flexure, as will be recalled, does not make it clear that the neutral axis will be normal to the plane of the loads only when the plane of the loads is a plane of symmetry of the section, or, more generally speaking, includes a principal axis of inertia of the section. To be sure, this condition does obtain in most cases in practice, but by no means in all, and errors have crept into some of our best books on structural design from a failure to realize the limitations of the familiar analysis. Occasional writers devote a moment's attention to what they call unsymmetrical bending, but, as far as the writer has observed, they always use the complicated method of the principal axes and moments of inertia. Accordingly, this paper is presented, not only on account of the scientific importance of the subject, but also in the hope that it may be of use to practitioners in actual design. The topic treated in this paper has, in recent years, been given much attention by various German writers, particularly Professors Müller-Breslau, Mohr, and R. Land, to whose work the writer renders most appreciative acknowledgment for indispensable aid in his studies. Some years ago the writer published a paper* on the subject, adhering for the most part to the German methods of deduction. But he has since devised methods which, it is believed, will be found much more natural, and they are offered herein with the conviction that some of the most serious complications, and perhaps all the avoidable ones, have been eliminated.

The important equation numbered 7 appeared independently, and as a result of different forms of deduction, in the third edition of Müller-Breslau's *Graphische Statik der Baukonstruktionen,* and in the writer's paper just cited. It appears herein deduced in still a third way. Plate V and Equations 11, 15, 16, and 18 are believed to have never been published before. What is called herein the *S*-polygon (perhaps the most useful of the results) is a modification of Land's "*W*-Fläche."

Sections 2 to 8 include all that is essential for a working knowl-

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* *Journal* of the Association of Engineering Societies, May, 1902.

edge of the subject. Sections 9 to 11 contain tributary matter of a relatively academic nature.

2. Statement of the Problem.—The problem to be solved may be stated in precise terms as follows:

> *Given:* A straight bar of uniform section subject to loads in any plane which includes the longitudinal axis of the body.*
>
> *Required:* I. The intensity of the normal stress at any point of a given right section;
>
> II. The extreme values of the normal stress intensity (called commonly extreme fiber stresses) in the given section;
>
> III. Quick practical means for computing these extreme fiber stresses in all cases;
>
> IV. Similar means, if possible, for indicating which of the familiar rolled-steel sections is to be selected to resist any case of general flexure in order to keep the extreme fiber stresses within prescribed limits.

The satisfaction of the first requirement will clear the way for the others. The latter, though, mathematically, corollaries to the first, are to the practitioner of paramount importance.

It is to be understood that the section of the body may or may not have an axis of symmetry, and that the forces are not too great to permit the usual assumption of linear distribution of stress. Extreme cases of irregularity of cross-section, which would interfere with the applicability of this assumption, might, perhaps, be imagined, but no section likely to be used in practice is believed to be of this sort.

3. Outline of Method of Procedure.—The external forces on the whole bar being in equilibrium, the force transmitted past the given section of the bar is the resultant, *R*, of all the forces on one side of the section—an equal, opposite and coincident resultant coming, of course, from the other side and together producing at the same time equilibrium of the body, and stress at the section.

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*This limitation of the location of the loads eliminates torsion—a stress which would not affect the problem, should it exist—and thus helps to fix ideas at the outset.

If the given forces should not be in a plane, as just limited, general flexure would be due only to their components in such a plane, and only such components would enter the problem. The above statement of the data, therefore, is actually general in spite of appearances to the contrary.

Suppose (Fig. 1) the point where R intersects the plane of the section be called K, distant q from the center of gravity, G, of the section. R may be resolved into two components at K, one normal to the section, the other tangential and passing through G. Only the former affects the problem; and it alone, under the designation, N, will henceforth be considered.

Statically speaking, the problem is to resolve N into an infinite number of non-co-planar parallel forces. Pure statics being unequal to the task, the necessary aid is found in the principle of linear distribution of stress, a principle now universally accepted as valid for moderately loaded beams of wood and metal. Stating this prin-

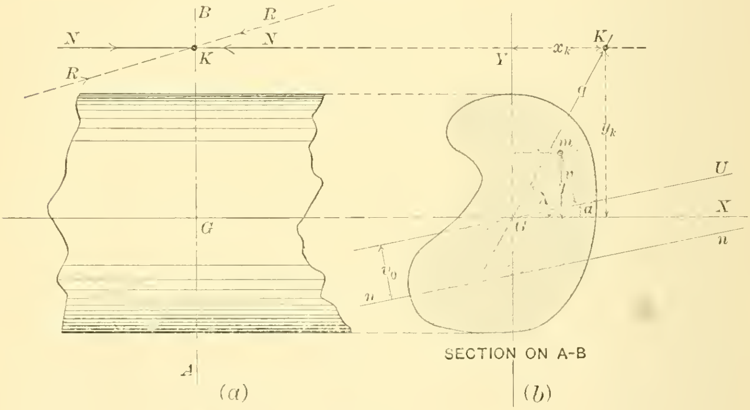


FIG. 1.

ciple and the conditions of equilibrium in algebraic language, there result four simultaneous equations, the solution of which yields the required quantity, stated in terms of the given quantities. This solution once obtained, it will receive further study, with a view to facilitating its application.

4.—Algebraic Statement of the Conditions.—Let the beam have any section, as that of Fig 1(b), referred to any convenient pair of rectangular axes, $G\ X$ and $G\ Y$, the origin, G, being the center of gravity of the section. Let f be the required stress-intensity at m, any infinitesimal portion of the section, distant x and y from $G\ Y$

and $G X$, respectively. The point, K, the co-ordinates of which will be called x_k, y_k, being, as above stated, the intersection of R with the section, and distant q from G, call λ the inclination of $G K$ to $G X$. Let $n\ n$ be the locus of points where f is zero (that is, the neutral axis) inclined to $G X$ by the angle, α. $G U$ is a line drawn through G parallel to $n\ n$. The distance between $n\ n$ and $G U$ will be called v_o.

The algebraic statement of the principle of linear distribution may conveniently be written

$$f = f_o - \frac{f_o}{v_o} v,$$

in which f_o is the constant value of f at all points of $G U$, and v is the distance of m from $G U$. The lengths, v and v_o, are + or — according as they are measured from $G U$ toward or away from K. Accordingly, f_o and $\frac{f_o}{v_o}$ are placed in the equation with opposite signs, and + is selected for the former. Then, as will be seen from Equation 8, a positive result for f will indicate that the stress at m is of the same sign as if N were applied at G, and *vice versa*. Referring m to $G X$ and $G Y$, by substituting $y \cos. \alpha - x \sin. \alpha$ for v, the equation becomes

$$f = f_o - \frac{f_o}{v_o} (y \cos. \alpha - x \sin. \alpha) \ldots\ldots\ldots\ldots\ldots (1)$$

Here, f_o, v_o, and α are unknowns, and three more equations are required for their evaluation. The conditions of equilibrium are now brought into use.

Expressing the infinitesimal area, m, as $d\, x\, d\, y$, an unbalanced single force will be precluded under the conditions if

$$\int\int f\, d\, x\, d\, y = N \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

and, similarly, an unbalanced couple will be precluded if

$$\int\int x f\, d\, x\, d\, y = N x_k \ldots\ldots\ldots\ldots\ldots (3)$$

and

$$\int\int y f\, d\, x\, d\, y = N y_k \ldots\ldots\ldots\ldots\ldots (4)$$

With these three equations the unknowns can be eliminated from Equation 1, and the expression for f will be left in proper shape for discussion and use.

5. *Deduction of the Desired Equations.*—Substituting* the value of f from Equation 1 in each of the Equations 2, 3 and 4, and noting that

$\int\int dx\,dy = A$, the area of the section,

$\int\int x\,dx\,dy = 0$, since $G\,Y$ is a gravity axis,

$\int\int y\,dx\,dy = 0$, since $G\,X$ is a gravity axis,

$\int\int y^2\,dx\,dy = I_x$†, the moment of inertia of the section referred to $G\,X$,

$\int\int x^2\,dx\,dy = I_y$†, the moment of inertia of the section referred to $G\,Y$,

$\int\int xy\,dx\,dy = J$†, the product of inertia of the section referred to $G\,X$ and $G\,Y$.

---

* This substitution being a highly important step, the algebraic reduction is stated as follows:

Equation 2, with $f_o - \frac{f_o}{v_o}(y \cos. a - x \sin. a)$ substituted for f, becomes

$$\int\int \left\{ f_o - \frac{f_o}{v_o}(y \cos. a - x \sin. a) \right\} dx\,dy = N,$$

that is,

$$f_o \int\int dx\,dy - \frac{f_o}{v_o} \cos. a \int\int y\,dx\,dy + \frac{f_o}{v_o} \sin. a \int\int x\,dx\,dy = N.$$

Inserting values of the integrals stated above, this becomes

$$A\,f_o = N,$$

whence Equation 5.

Equation 3, upon substitution of $f_o - \frac{f_o}{v_o}(y \cos. a - x \sin. a)$ for f, becomes

$$\int\int \left\{ f_o - \frac{f_o}{v_o}(y \cos. a - x \sin. a) \right\} x\,dx\,dy = N\,x_k,$$

that is,

$$f_o \int\int x\,dx\,dy - \frac{f_o}{v_o} \cos. a \int\int xy\,dx\,dy + \frac{f_o}{v_o} \sin. a \int\int x^2\,dx\,dy = N\,x_k.$$

Inserting the stated values of the integrals as before, this becomes

$$-\frac{f_o\,J \cos. a}{v_o} + \frac{f_o\,I_y}{v_o} \sin. a = N\,x_k,$$

or

$$-\frac{f_o}{v_o}(J \cos. a - I_y \sin. a) = N\,x_k.$$

whence

$$-\frac{f_o}{v_o} = \frac{N\,x_k}{J \cos. a - I_y \sin. a},$$ which is part of Equation 6.

Similarly, by substituting Equation 1 in Equation 4 it falls out that

$$-\frac{f_o}{v_o} = \frac{N y_k}{I_x \cos. a - J \sin. a},$$

which completes Equation 6.

† Computation of I_x, I_y, and J.—The computation of I_x, and I_y involves, in practice, only the application of the familiar, $I' = I_o + A\,h^2$. For all the structural shapes, the

it will appear that

$$f_o = \frac{N}{A} \quad \text{.........................} (5)$$

(showing that the unit stress at the center of gravity is always the same as it would be if N were applied there), and that

$$\frac{f_o}{v_o} = \frac{N\, x_k}{J \cos. \alpha - I_y \sin. \alpha} = \frac{N\, y_k}{I_x \cos. \alpha - J \sin. \alpha} \quad \text{........} (6)$$

Solving Equation 6 yields, noting that $\frac{y_k}{x_k} = \tan. \lambda$.

$$\left.\begin{aligned} \tan. \alpha &= \frac{I_x - J \tan. \lambda}{J - I_y \tan. \lambda} \\ &= \frac{I_x \cot. \lambda - J}{J \cot. \lambda - I_y} \\ &= \frac{I_x \cos. \lambda - J \sin. \lambda}{J \cos. \lambda - I_y \sin. \lambda} \end{aligned}\right\} \quad \text{...............} (7)$$

the various forms of the second member being convenient in special cases. The first two are usually the simplest to use, but either may become indeterminate on the substitution of special values of λ. If one of these fails in this way, the other or the third can be used.

Substituting Equations 5 and 6 in Equation 1, there results

$$f = \frac{N}{A} + \frac{N x_k (y - x \tan. \alpha)}{J - I_y \tan. \alpha} \quad \text{...............} (8)$$

or

$$f = \frac{N}{A} + \frac{N y_k (y - x \tan. \alpha)}{I_x - J \tan. \alpha} \quad \text{...............} (9)$$

Equations 8 and 9, if written in terms of q, will express f as the result of combining two quantities which represent, respectively, the contributions to f from a force, N, at G, and the couple, Nq, that is, from a force and couple into which any N at K can be resolved.

I_x and I_y are given outright by the handbooks. J is not so familiar a quantity and is not mentioned by the handbooks, yet its computation in practical cases is very easy. From the nature of its definition,

$$J = \int\int x\, y\, d\, x\, d\, y,$$

it is evident that, for a pair of axes, one of which is an axis of symmetry of the section, $J = 0$. It is also zero for the principal axes of inertia. In all ordinary cases it can be computed by the aid of the simple transformation formula, exactly analogous to $I' = I_o + A\, h^2$,

$$J' = J_o + A\, k\, h,$$

in which J' is the J of the section referred to any rectangular co-ordinates, $O\,X$ and $O\,Y$, k and h are co-ordinates of the center of gravity, G, of the section; J_o is the J referred to axes parallel to $O\,X$ and $O\,Y$, with origin at G. Numerical illustrations of the computation of J will be found in Sections 5 and 6. J will be positive or negative according as the preponderance of the section is in the first and third, or second and fourth quadrants

I_x, I_y, and J are all quantities of the fourth degree, and are expressed in biquadratic inches or biquadratic centimeters, etc.

Noting that x_k and y_k are q cos. λ and q sin. λ, respectively, Equations 8 and 9 become, accordingly,

$$f = \frac{N}{A} + \frac{N q (y - x \tan. \alpha) \cos. \lambda}{J - I_y \tan. \alpha} \dots\dots\dots\dots\dots (8')$$

and

$$f = \frac{N}{A} + \frac{N q (y - x \tan. \alpha) \sin. \lambda}{I_x - J \tan. \alpha} \dots\dots\dots\dots\dots (9')$$

Indeterminations in all these equations can be evaluated by the aid of trigonometric transformations such as were used in stating the second member of Equation 7, or by selecting between Equations 8 and 9, or between Equations 8′ and 9′.

The value of f is now stated in the way most convenient for the common practical problem, which is to find the maximum value of f, that is, the value of f for the m most remote from $n\ n$. This particular m will be the m most remote from $G\ U$ on the same side of $G\ U$ as K, as will be shown in Section 9. The routine in the solution of this problem will consist of:

(a) The solution of Equation 7;

(b) The identification of the most remote m, by sketching or drawing $G\ U$—a line through G the slope of which is tan. α.

(c) The substitution of the tan. α and x, y for this m in Equations 8 or 9.

Parts I and II of the requirements of the problem are now accomplished. A numerical example will be worked out by way of illustration, and then attention will be directed to a study of the equations just obtained, with a view to meeting requirements III and IV of the problem (Section 2).

It will be worth while, in passing, to combine Equation 7 with Equations 8 or 9, and with Equations 8′ or 9′, and record expressions for f in terms of known quantities, exclusively. They are found to be:

$$f = \frac{N}{A} + \frac{N (y_k I_y - x_k J) y + N (x_k I_x - y_k J) x}{I_x I_y - J^2} \dots\dots (10)$$

or

$$f = \frac{N}{A} + N_q \frac{(I_y \sin. \lambda - J \cos. \lambda) y + (I_x \cos. \lambda - J \sin. \lambda) x}{I_x I_y - J^2} . (10')$$

Numerical Example.—Problem.—A single 4 by 3 by ⅜-in. angle, as a member of a framework, is attached at each end to a ½-in. gusset

by a line of rivets of $2\frac{1}{4}$-in. gauge, through the 4-in. leg only. Suppose the direct stress is a thrust, P, acting in the line of the rivets and at mid-thickness* of the gusset. What is the maximum compressive unit stress in the angle?

Solution.—The solution consists of mere substitution in Equations 7 and 8 or 9, the determination of J offering the only unusual difficulty.

Showing the data in a sketch (Fig. 2) in which the section is referred to the most convenient axes, the section is divided into two

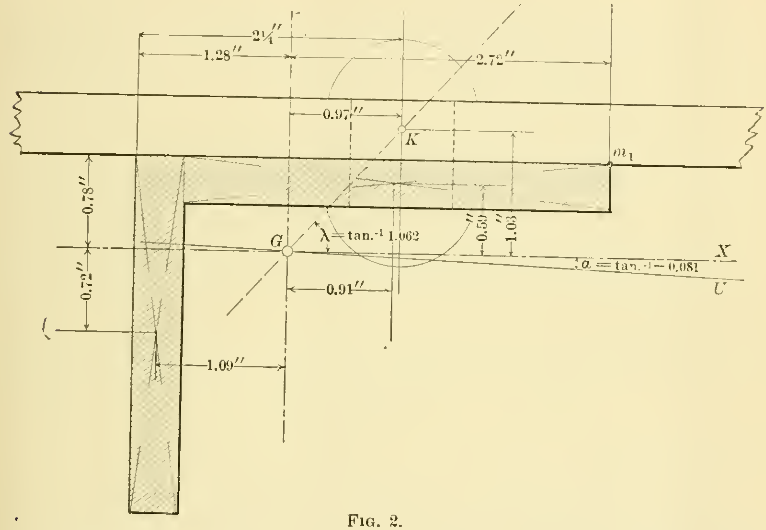


FIG. 2.

rectangles, whose centers of gravity are at (— 1.09, —0.72) and (0.91, 0.59), and whose areas are $3 \times \frac{3}{8} = 1.125$ and $3\frac{5}{8} \times \frac{3}{8} = 1.359$, respectively. Then, by $J' = J_o + A\,k\,h$, of the foot-note, p. 72, noting that J_o of both rectangles is zero,

$$J = 1.125 \times -1.09 \times -0.72 + 1.359 \times 0.91 \times 0.59 = 1.61 \text{ in.}^4$$

The Carnegie book gives $I_x = 1.92$ in.^{4}, $I_y = 3.96$ in.^{4}, $A = 2.48$ in.

Then, by Equation 7, noting that $\tan.\ \lambda = \dfrac{1.03}{0.97} = 1.062$,

$$\tan.\ \alpha = \frac{1.92 - 1.61 \times 1.062}{1.61 - 3.96 \times 1.062} = \frac{0.21}{-2.60} = -0.081.$$

* This problem is solved upon other assumptions as to the point of application of the thrust, at the close of Section 9.

$G\,U$, accordingly, is in the second and fourth quadrants, and the extreme fibers of the angle will be the most remote corners in the first and third quadrants, with m_1 as the fiber subject to maximum compressive stress. The co-ordinates of m_1 are (2.72, 0.78), and observing that $y_k = 1.03$, substitution in Equation 9 yields

$$f = \frac{N}{2.48} + N \frac{1.03\left(0.78 - \overline{2.72 \times - 0.081}\right)}{1.92 - 1.61 \times - 0.081}$$

$$= 0.40\,N + 0.50\,N = 0.90\,N, \text{ the required answer.}$$

That is, the effect of the eccentricity is to make the extreme stress more than double the average. If the direct stress, N, is 10 000 lb., $f_{max.}$ will be 9 000 lb. per sq. in. Equation 8, of course, would give the same result.

If the strut is so long as to deflect materially, the section at the point of maximum sidewise deflection will have a longer q and a different λ from that in the preceding solution. For such a case the method would be precisely as above, after the change in direction and length of $G\,K$ had been estimated—a process outside the scope of this paper.

6. Special Forms of Expressions for "f."—In pure normal stress, when q and x_k and y_k, all vanish, Equations 8, 9, and 10 reduce to

$$f = \frac{N}{A} \qquad \text{(10a)}$$

as they should.

In pure flexure, $N = 0$, $q = \infty$, and $N\,q$ is the value of the bending couple commonly written M. In this case Equations 8, 9 and 10, take the forms

$$f = \frac{M(y - x \tan. \alpha) \cos. \lambda}{J - I_y \tan \alpha.} \qquad \text{(8a)}$$

$$f = \frac{M(y - x \tan. \alpha) \sin. \lambda}{I_x - J \tan. \alpha} \qquad \text{(9a)}$$

$$f = M\,\frac{(I_y \sin. \lambda - J \cos. \lambda)\, y + (I_x \cos. \lambda - J \sin. \lambda)\, x}{I_x I_y - J^2} \qquad \text{(10b)}$$

Then, if $\lambda = 90°$, and $\alpha = 0°$, as it will if J is zero, these equations all reduce to the familiar but highly special

$$f = \frac{M y}{I_x} \qquad \text{(10c)}$$

The very common applicability of this equation arises from the fact that the rectangular beams and beams with I, C, or T-sections are naturally referred to rectangular axes for which J is zero, and are loaded in a plane containing one of these axes.

Numerical Examples.—1.—A 5 by $3\frac{1}{4}$ by $\frac{1}{2}$-in. Z-bar acts as a purlin on a roof having a slope of 30°, its top flange projecting toward the ridge. It supports vertical loads which cause a maximum flexure of M inch-pounds. Required, the extreme fiber stresses.

Solution.—Showing data, and taking axes as in Fig. 3, I_x and

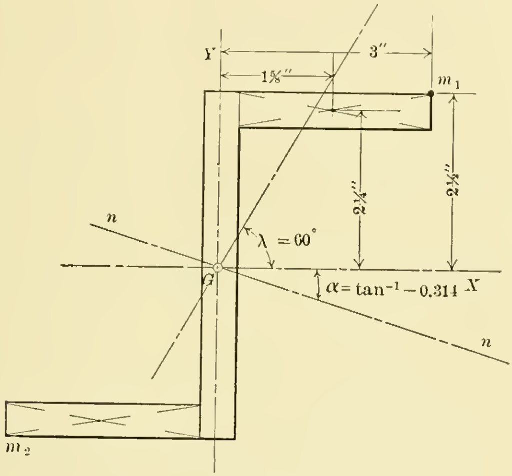


Fig. 3.

I_y are given by steel handbooks as 19.19 in.^{4} and 9.05 in.^{4}, and J, computed in a manner similar to that of the angle iron of the preceding section, is found to be $2(2.75 \times 0.5 \times 2.25 \times 1.625) = + 10.05$ in.^{4}

By Equation 7, tan. λ being tan. $60° = 1.732$,

$$\tan.\ \alpha = \frac{19.19 - 10.05 \times 1.732}{10.05 - 9.05 \times 1.732} = -0.317$$

The extreme fibers are located accordingly at m_1 and m_2 the co-ordinates of which are $\pm$ 3.0 and $\pm$ 2.5. Using the values for m_1, and substituting in Equation 9a.

$$f = \frac{(2.5 - 3.0 \times - 0.317) \times 0.866}{19.19 - 10.05 \times - 0.317} M = 0.134\ M \text{ lb. per sq. in.}$$

The f for m_2 would differ from this only in sign, and $\pm 0.134\ M$ is the required answer.

2.—Suppose the Z-bar of the preceding case replaced by a 6 by 12-in. rectangle with the 12-in. side normal to the roof slope. Find the fiber stresses.

Solution.—Taking $G\ X$ parallel to the short side, J is zero, and I_x and I_y are 864 and 216 in.^{4}, respectively. Hence, by Equation 7.

$$\tan.\ \alpha = \frac{864}{-216 \times 1.732} = -2.31$$

The extreme fibers are ± 3.0 and ± 6.0. Inserting in Equation 9a

$$f = \pm \frac{(6 - \overline{3 \times -2.31}) \times 0.866}{864} M = \pm 0.013\ M \text{ lb. per sq. in., the}$$

required answer.

7. *The S-polygon of a Section.*—As has been observed, any case of flexure may be treated as a combination of direct normal stress and pure flexure, either of which may, of course, be zero. The treatment of the pure flexure is the only part offering much difficulty. The difficulty with pure flexure is specially great when the extreme value of f is to be determined, and the proper values of x and y, substituted in Equation 10b, have to be computed in advance. The diminution of the difficulty in this most important practical problem is the next field of inquiry.

Inspection of Equations 8a, 9a and 10b reveals that the f for any m may be obtained by dividing M by a quantity in which, for a given λ, the only variables are the co-ordinates of m. This quantity, in its most general form, is, from Equation 10b

$$S = \frac{I_x\, I_y - J^2}{(y\, I_y - x\, J)\ \sin.\ \lambda + (x\, I_x - y\, J)\ \cos.\ \lambda} \ldots\ldots\ldots (11)$$

a quantity which might be called the flexure modulus of the section for the point, x, y, or, more briefly, the section modulus for x, y. Special values of it for extreme positions of m and for special values of λ are what are given in the steel manufacturers' handbooks in the case of I-beams, channels and tees as the section moduli of the shapes concerned. In this paper it will be referred to simply as S, occasionally with a special suffix.

S is a quotient obtained by dividing a quantity of the eighth degree by one of the fifth. It is, therefore, of the third degree. It may

be looked upon as the statical moment of an area.* Thus, only can M divided by S yield a stress-intensity. In the special case in which $\lambda = 90°$, and J is zero, S takes the familiar form

$$S_a = \frac{I_x}{y} \dots\dots\dots\dots\dots\dots (12)$$

Returning to the examination of Equation 11, it at once appears that, for a fixed m and varying λ, that equation is the polar equation of a straight line, that is, a line the radius vector of which, at any inclination, λ, would be proportional to the S for that λ. That is, the S-line, if it may be so called, is the simplest possible graphical exhibit for the values of S for any one m for all values of λ. This at once suggests the determination of the S-lines for all points of the section which might be extreme fibers of the section. As will be seen in what follows, these S-lines will define a surface which may be called the S-polygon of the section. With the S-polygon given, merely drawing a line through G with the inclination, λ, defines two radii vectores, S' and S'', measured from G toward and away from K, respectively. S' and S'' being measured to the proper scale, the most important values of f in the most difficult case can be had by mere substitution in

$$f' = \frac{N}{A} + \frac{Nq}{S'} = \frac{N}{A} + \frac{M}{S'} \dots\dots\dots\dots\dots\dots (13)$$

$$f'' = \frac{N}{A} + \frac{Nq}{S''} = \frac{N}{A} - \frac{M}{S''} \dots\dots\dots\dots\dots\dots (14)$$

which are perfectly general expressions for the extreme values of f, Of course, in pure flexure, N vanishes, and they become

$$f' = \frac{M}{S'} \dots\dots\dots\dots\dots\dots (13')$$

$$f'' = \frac{M}{S''} \dots\dots\dots\dots\dots\dots (14')$$

8.—Construction of the S-polygon.—Referring Equation 13 to rectangular co-ordinates by substituting x and y for S cos. λ, and S sin. λ, respectively, we have

$$y = \frac{x_m I_x - y_m J}{x_m J - y_m I_y} x - \frac{I_x I_y - J^2}{x_m J - y_m I_y} \dots\dots\dots\dots (15)$$

a line which is parallel to the neutral axis (Equation 17 in Section 9) for a K at x, y, on the same side of G with K (that is, on the opposite side of G from n n), and, if the same units be selected to rep-

* The units in which it is expressed may be called inch square-inches, following the analogy with inch-pounds. Inch square-inches will be abbreviated to inches[3] or in.[3]

resent third-degree units as to represent simple length, A times as far from G as $n\ n$ is.

Of course, the values of x and y, for all points on the perimeter of the given section could be substituted for (x_m and y_m) in Equation 15. the corresponding lines plotted, and the space within all these lines taken as the S-polygon. This might have to be resorted to for parts of the section bounded by curved lines, but it can be done more easily in the case of the familiar rolled steel shapes. In these cases the extreme m's will all lie upon a small number of straight lines which bound, but do not, even if produced, cross the section. These lines will constitute what may be called the circumscribing polygon. The S-polygon will have a side corresponding to each apex of the circumscribing polygon. These sides will be S-lines, intersecting and forming apices of the S-polygon whenever λ has such a value as will cause two adjacent apices of the circumscribing polygon to be extreme fibers at the same time. This will happen, of course, whenever the neutral axis is parallel to the side of the circumscribing polygon.

Locating and connecting the apices will usually be the most convenient way of constructing the S-polygon. The co-ordinates of an apex of the S-polygon can be determined by substituting the co-ordinates (x_a, y_a) and (x_b, y_b) of two successive apices, A and B, of the circumscribing polygon in Equation 15, and solving the two resulting equations for the co-ordinates of the intersection of the resulting S-lines. Performing this operation will yield for the co-ordinates (x_{ab}, y_{ab}) of the apex of the S-polgyon corresponding to the side $A\ B$,

$$\left.\begin{aligned} x_{ab} &= \frac{(x_a - x_b)\ J - (y_a - y_b)\ I_y}{x_a\ y_b - x_b\ y_a} \\ y_{ab} &= \frac{(x_a - x_b)\ I_x - (y_a - y_b)\ J}{x_a\ y_b - x_b\ y_a} \end{aligned}\right\} \qquad (16)$$

If, as very commonly happens, the side of the circumscribing polygon is parallel to an axis of reference, these expressions become materially simplified. If, for instance, it is parallel to the X-axis, $y_a = y_b$, and Equation 16 becomes

$$\left.\begin{aligned} x_{ab} &= \frac{J}{y_a} \\ y_{ab} &= \frac{I_x}{y_a} \end{aligned}\right\} \qquad (16')$$

If the line is parallel to the Y-axis, $x_a = x_b$, and Equation 16 becomes

$$\left.\begin{aligned} x_{ab} &= \frac{I_y}{x_a} \\ y_{ab} &= \frac{J}{x_a} \end{aligned}\right\} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16'')$$

The actual S-polygon, as a fixed geometrical property of a section, and one which can be constructed with great ease any time (see numerical examples below) for the simpler sections, and can be constructed once for all and kept on file for the more difficult sections, is presented as a satisfaction of the requirements of III and IV of Section 2.

The actual methods of computation and use of the S-polygon will be illustrated in the following numerical examples.

Numerical Examples.—Required the S-polygon for each of the four following sections, viz.:

(*a*) 6 by 12-in. rectangle;

(*b*) 8-in. 18-lb. I-beam;

(*c*) 10-in. 20-lb. channel;

(*d*) 5 by ½-in. Z-bar.

Solution.—The natural axes of reference for all these shapes are the gravity axes parallel to the principal dimensions, and shown in Figs. 4, 5, 6, and 7. In each of the first three sections at least one of the axes is an axis of symmetry and J will be zero, and, consequently, the simple Equations 16′ and 16″ will suffice. For the Z-bar, however, Equation 16, in its general form, will be needed. The sides and apices of the S-polygons will be designated with the lower-case letters of the corresponding apices and sides of the circumscribing polygon.

The four shapes will now be taken up in order.

(*a*)—The given 6 by 12-in. rectangle is shown as $A\ B\ C\ D$ in Fig. 4.

$$I_x = \frac{b\,d^3}{12} = \frac{6 \times 12 \times 12 \times 12}{12} = 864 \text{ in.}^4;$$

$$I_y = \frac{b^3\,d}{12} = \frac{6 \times 6 \times 6 \times 12}{12} = 216 \text{ in.}^4;$$

$$J = 0.$$

It will suffice to compute the co-ordinates of the apices, $a\,b$ and $b\,c$; symmetry will locate the other two, $c\,d$ and $a\,d$.

The co-ordinates of A, B, and C are $(-3, 6)$, $(3, 6)$ and $(3, -6)$. These are the (x_a, y_a), (x_b, y_b), etc., of Equations 16, 16′, and 16″.

By Equation 16′, $x_{ab} = \dfrac{J}{y_a} = \dfrac{0}{6} = 0$; $y_{ab} = \dfrac{I_x}{y_a} = \dfrac{864}{6} = 144 \text{ in.}^3$

By Equation 16″, $x_{bc} = \dfrac{I_y}{x_b} = \dfrac{216}{3} = 72 \text{ in.}^3$; $y_{bc} = \dfrac{J}{x_b} = \dfrac{0}{3} = 0.$

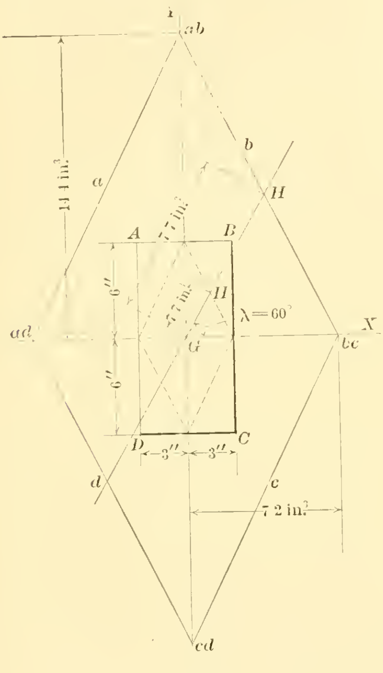


Fig. 4.

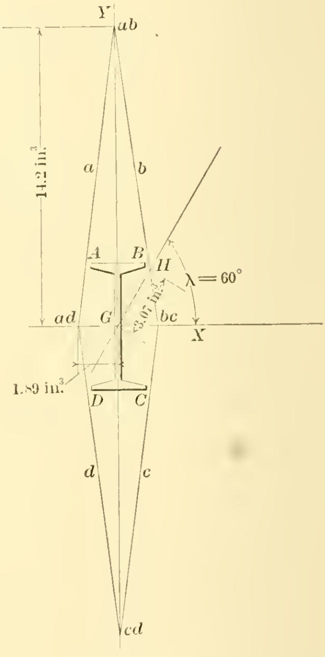


Fig. 5.

The four apices are, accordingly, $(0, \pm 144)$ and $(\pm 72, 0)$, and are plotted in Fig. 4 to an arbitrarily selected scale.

Remarks.—The sides of this S-polygon are parallel to the diagonals of the rectangle, and an S-polygon, therefore, could be constructed by merely connecting the middle points of adjacent sides of the rectangle. Such a polygon is shown dotted in Fig. 4. The scale of this S-polygon is readily determined by computing the numerical value, $\dfrac{b\,d^2}{6}$, of its vertical semi-diagonal.

The apices are seen to be on the axes at distances from G which represent the familiar section moduli in the planes of the axes along which they are laid off. Consequently, in dealing with rectangles in general, or similar simple sections, no formal use of Equations 16′ and 16″ is necessary. The section moduli once known, they are laid off to convenient scale along the proper axes, and the apices thus determined are connected. This would apply to any section referred to rectangular gravity axes for which J is zero. This method will be exemplified in the next two cases.

(*b*)—For the 8 in., 18-lb. I-beam, the steel handbooks give the section modulus for the axis, $G\ X$, as 14.2 in.^{3} The other is found by dividing the I_y by half the flange width—both given by the handbooks, that is, $\frac{3.78}{2.0} = 1.89$ in.^{3} Laying off the 14.2 upward and downward from G, and the 1.89 to the right and left of G, the S-polygon of Fig. 5 is established.

(*c*)—For the 10-in., 20-lb. channel, the two section moduli, 15.7 and 1.34 in.^{3}, given by the handbooks must be supplemented by a third one, since $G\ Y$ is not an axis of symmetry. This third one is I_y divided by the distance from G to the back of the channel. Using the handbook data, this is found to be $\frac{2.85}{0.61} = 4.67$ in.^{3} Laying off 15.7 upward and downward from G, 1.34 from G along $G\ X$ and in the same direction as the projecting flanges, and 4.67 along $G\ X$ in the opposite direction, the S-polygon of Fig. 6 is located.

(*d*)—For the 5 by $\frac{1}{2}$-in. Z-bar, the handbooks furnish $I_x =$ 19.19 in.^{4}, $I_y = 9.05$ in.^{4}, and, in Section 7, J was found to be 10.05 in.^{4} J will be positive for the section as drawn in Fig. 7. The three apices, $a\ b$, $b\ c$, and $c\ d$, will be located by computation, and the other three, $d\ e$, $e\ f$, $f a$, will follow by symmetry. The coordinates of A, B, C, and D are (— 0.25, 2.50), (3.0, 2.50), (3.0, 2.0), and (0.25, — 2.50)

By Equation 16′,

$$x_{ab} = \frac{10.05}{2.50} = 4.02 \text{ in.}^3; \quad y_{ab} = \frac{19.19}{2.50} = 7.68 \text{ in.}^3.$$

By Equation 16″,

$$x_{bc} = \frac{9.05}{3.0} = 3.02 \text{ in.}^3; \quad y_{bc} = \frac{10.05}{3.0} = 3.35 \text{ in.}^3.$$

By Equation 16,

$$x_{cd} = \frac{(3.0 - 0.25)10.05 - (2.0 - 2.5)9.05}{3.0 \times -2.5 - 0.25 \times 2.0} = 1.63 \text{ in.}^3;$$

$$y_{cd} = \frac{(3.0 - 0.25)19.19 - (2.0 - 2.5)10.05}{3.0 \times -2.5 - 0.25 \times 2.0} = -0.94 \text{ in.}^3.$$

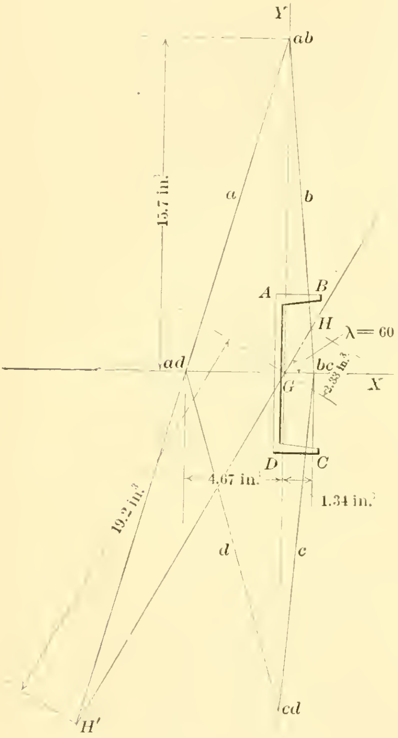


FIG. 6.

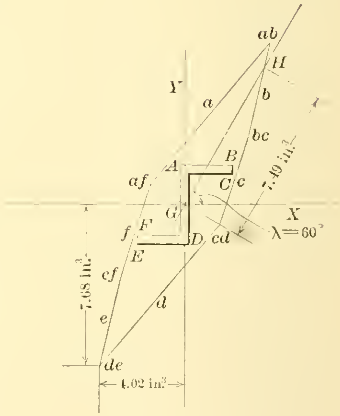


FIG. 7.

These and the same with signs reversed establish the desired S-polygon plotted in Fig. 7. Similarly, any section can be treated.

9. Examples of the Use of the S-Polygon.—The following numerical examples will illustrate some of the uses of the S-polygon.

Problem 1.—Find the extreme fiber-stress in each of the four sections of Figs. 4 to 7, assuming each to be acting as a purlin upon a roof inclined 30° to the horizon and set with its G X-axis parallel to the roof slope. Assume a flexure of M, in pounds, in a vertical

plane, and assume that each purlin is set so as to make B the highest corner.

Solution.—Draw a line with a slope, $\lambda = (90° - 30°) = 60°$ (representing the trace of the plane of loads) through G in each S-polygon of Figs. 4 to 7, and scale off the distance, $G\ H$, along this line to the S-polygon perimeter, taking the shorter of the two lengths when there is a difference. The results for the four sections are as dimensioned 77, 3.07, 2.33, and 7.49 in.^{3}, respectively. The results, accordingly, are by Equation 13, N being zero:

For the rectangle:

$$f_{max.} = \pm \frac{M}{77} = \pm\ 0.013\ M \text{ lb. per sq. in.}$$

For the 8-in., 18-lb. I-beam:

$$f_{max.} = \pm \frac{M}{3.07} = \pm\ 0.325\ M \text{ " " " "}$$

For the 10.-in., 20-lb. channel:

$$f_{max.} = + \frac{M}{2.33} = +\ 0.428\ M \text{ " " " "}$$

For the 5 by ½-in. Z-bar:

$$f_{max.} = \pm \frac{M}{7.49} = \pm\ 0.134\ M \text{ " " " "}$$

Here are verified the results of the numerical example of Section 6. Incidentally, note the great economy (for such a load) of the Z-bar over the heavier channel and I-beam.

Problem 2.—In each of the sections in the preceding problem, which are the fibers subject to the stresses given?

Solution.—In the rectangle and I-beam, H might be taken on either the b-line or the d-line. Hence, B and D are both extreme fibers in these cases. In the channel, similarly, B is indicated, and in the Z-bar, B and E.

Problem 3.—In the channel of Problem 1, what is the magnitude and nature of the stress at the point, A, under the given load?

Solution.—Extending $G\ H$ to the intersection, H', with the a-line produced, $G\ H'$ is scaled off as 19.2 in.^{3} Hence, $f_A = \frac{M}{19.2} = 0.052\ M.$ To determine whether it is tension or compression, observe that N, though zero in amount, may be treated as a compressive force at infinity on $G\ H$ above G, or as a tensile force at infinity below G.

Assume the former, then H' is on the opposite side of G from K, and f_A, accordingly (Section 7), is of opposite sign to N, and, therefore, is tensile. The same result follows, of course, from the opposite assumption.

Problem 4.—In the Z-bar of Fig. 7, what is the most favorable plane of loading, and what will then be the position of the neutral axis?

Solution.—The most favorable plane of loading is obviously that yielding the maximum S. This, by inspection of the figure, would be a plane the trace of which would pass through $a\ b$ and $d\ e$, and have for a slope $\tan.^{-1} \frac{7.68}{4.02}$. This trace, passing through $a\ b$ and $d\ e$, indicates that A and B, as well as D and E, are then all extreme fibers. Hence, the neutral axis is then parallel to the flanges, $A\ B$ and $D\ E$.

Similarly, the least favorable plane of loads might be located; obviously, it will be perpendicular to the a-line and d-line.

Problem 5.—Through what extremes would $f_{max.}$ pass, if the Z-bar of Problem 1 were to make a complete revolution about its longitudinal axis, the load remaining vertical?

Solution.—Scaling off the extreme values of S indicated in the preceding problem, they are found to be 8.68 and 1.85 in.^{3} The extremes of $f_{max.}$ are $\frac{M}{8.68} = 0.116\ M$, and $\frac{M}{1.85} = 0.540\ M$.

Problem 6.—What is the least favorable plane of loads for the rectangle of Fig. 4?

Solution.—Minimum S will occur for planes normal to either diagonal of the rectangle.

Problem 7.—What plane of loads will make the neutral axis vertical in the Z-bar of Fig. 7:

Solution.—If the neutral axis is vertical, $B\ C$ and $E\ F$ will be the extreme fibers, and the plane of loads must pass through $b\ c$ and $e\ f$.

Problem 8.—A channel is to be selected to act as a purlin; trusses 10 ft. on centers; slope of roof 30° to horizontal; purlins 6 ft. apart in plan; loads, dead 10 lb. and snow 20 lb. per sq. ft. of plan, and wind 25 lb. per sq. ft. of roof surface; fiber stress not to

exceed 16 000 lb. per sq. in. for the worst combination, considering (*a*) that the maximum snow and wind may act simultaneously; (*b*) that they may not so act.

Solution.—The loads per linear foot upon the purlin for the three

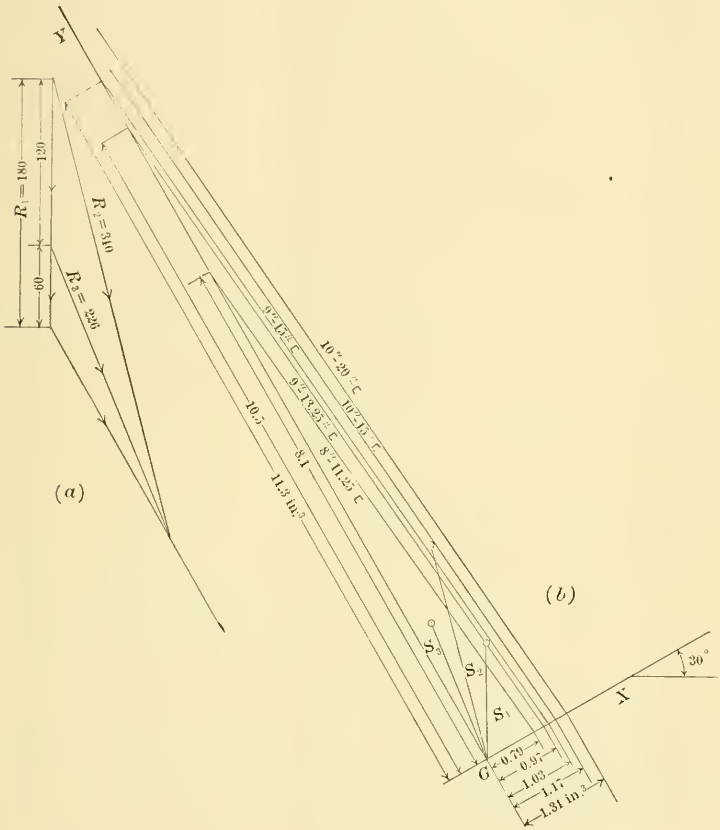


Fig. 8.

loads are, noting that the distance between purlin centers on the slope is 6.93 ft.,

$w_d = 6 \quad \times 10 = \quad 60$ lb. per lin. ft., vertical,

$w_s = 6 \quad \times 20 = 120$ " " " " "

$w_w = 6.93 \times 25 = 173.2$ " " " " normal to roof slope.

Fig. 8*a* is a force-magnitude polygon showing the important result-

ants. R_1, R_2 and R_3 in magnitude and direction. They are the resultants of w_d and w_s, of w_d, w_s, w_w, and of w_d and w_w, respectively. They give rise to three different flexures, M_1, M_2 and M_3, in their respective planes, and call for three different values of S: S_1, S_2 and S_3, in those planes.

$$S_1 = \frac{M_1}{16\,000} = \frac{180 \times 10 \times 10 \times 12}{8 \times 16\,000} = 180 \times 0.0094 = 1.69 \text{ in}^3.$$

Similarly, S_2 and S_3 are 3.19 and 2.12 in.^{3}, respectively.

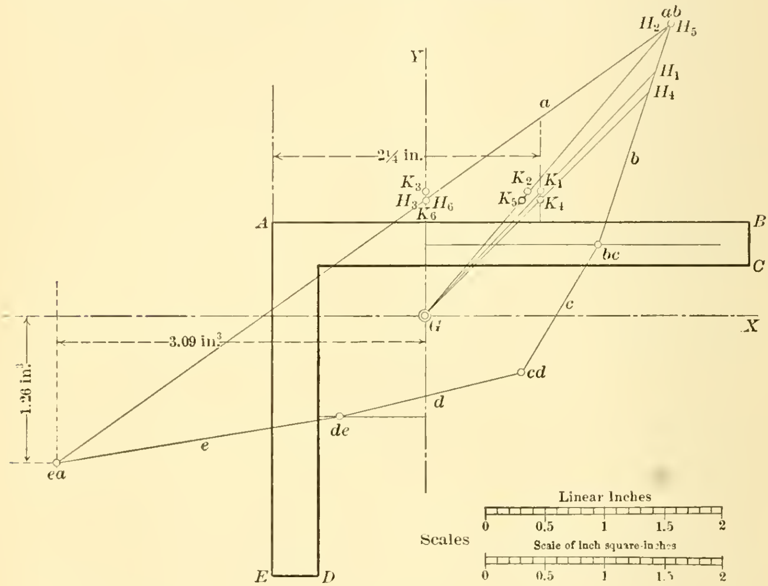


FIG. 9.

On inspection of Fig. 6 it is seen that a single side (that pertaining to B) of the S-polygon for each channel will suffice. Accordingly, this side is plotted for a promising list of channels, using only such data as the handbook furnishes. S_1, S_2 and S_3 are laid off from G parallel to the resultants to which they respectively correspond. This work is shown in Fig 8*b*. Condition (*a*) calls for a channel which can furnish S_1 and S_2, and condition (*b*) requires S_1 and S_3. A glance at the sketch shows that for (*a*) a 10-in. 15-lb. channel is required, and for (*b*) a 9-in. 13.25-lb. channel. The latter is so close a fit that one of the next two larger sizes might well be preferred, in view of the sensitiveness of the section to a slight change in position.

Problem 9.—Suppose the 4 by 3 by $\frac{3}{8}$-in. angle of the example at the close of Section 5 to be exposed to a thrust, P (= 18 000 lb.) applied successively at the six different K's, shown in Fig. 9. The first three are at mid-thickness of a $\frac{1}{2}$-in. gusset, and the second three similarly on the center plane of a $\frac{3}{8}$-in. gusset. In each set, the K's are, respectively, at gauge $2\frac{1}{4}$ in., at the most favorable gauge, and opposite G. Required the $f_{max.}$ for each case.

Solution.—Here are really six problems, each equal in complexity to that at the close of Section 5. In fact, the first one is that very problem, here to be solved again for comparison. The S-polygon simplifies matters so greatly that the six problems will be taken up together. It is merely a question of getting the M's and S''s for substitution in Equation 13. S' is, in each case, $G\ H$, as usual, and $M = P\ q$, where q is $K\ G$. The six q's and S''s are scaled from Fig. 9, where the section and its S-polygon are plotted, and the substitution is carried out in Table 1.

TABLE 1.

Case.	q.	S'.	$\frac{M}{S'} = \frac{P\ q}{S'}$.	$\frac{P}{A}$	$f_{max} = \frac{P}{A} + \frac{M}{S'}$	Location of f_{max}.
K_1	1.41 in.	2.82 in.^{3}	9 000	7 200	16 200	B
K_2	1.35 "	3.21 "	7 600	7 200	14 800	$A\ B$
K_3	1.03 "	0.97 "	19 100	7 200	26 300	A
K_4	1.37 "	2.66 "	9 300	7 200	16 500	B
K_5	1.27 "	3.21 "	7 100	7 200	14 300	$A\ B$
K_6	0 97 "	0.97 "	18 000	7 200	25 230	A

The location of the fiber, subject to $f_{max.}$ in each case, is indicated by the letter of the S-line on which H falls, and is recorded in the table as a matter of interest.

The fluctuation of f is seen to be considerable with these changes in the position of K. In an actual frame these variations would doubtless be less serious, but would be diminished only at the expense of secondary stresses in the neighboring members. These stresses would be of a flexural or torsional character very difficult to estimate.

Section 10.—Concerning S-Polygons of Specially Important Sections.—Table 2 gives the co-ordinates of the apices of the S-polygons for the standard American **Z**-bars, and Plate V shows these same S-polygons plotted to scale. It is believed that the reader will be able from the foregoing to establish the S-polygons

for the standard channels so easily as to make it inadvisable to take up the space to show them here. For I-beams no computation is needed beyond what is given in the steel handbooks. Thus would be covered the most important purlin shapes. The writer has never established the S-polygons for angles, but rests content in that they are relatively of minor importance.

TABLE 2.—APEX CO-ORDINATES FOR THE S-POLYGONS OF THE AMERICAN STANDARD Z-BARS, EXPRESSED IN INCHES.[3]

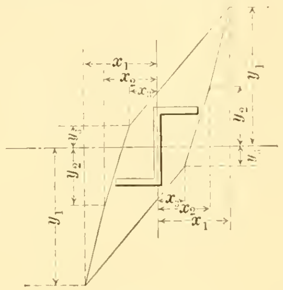
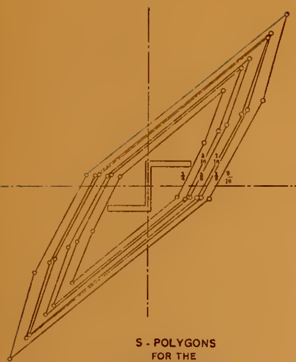


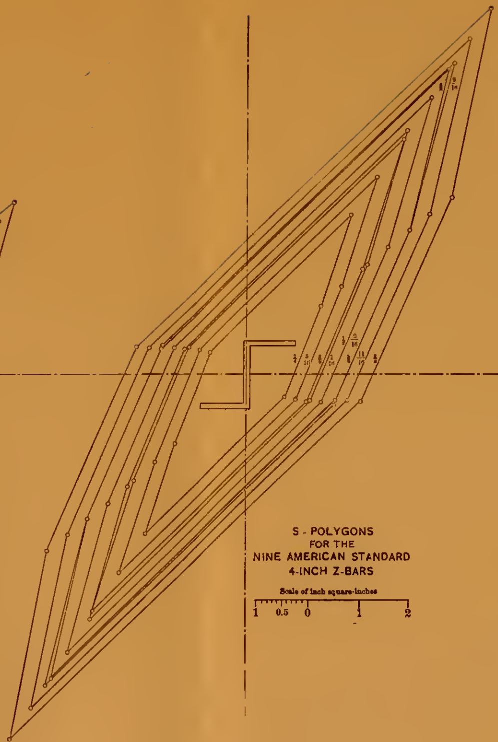
FIG. 10.

Section.	x_1	y_1	x_2	y_2	x_3	y_3
$3 \times 2\frac{11}{16} \times \frac{1}{4}$	1.50	1.92	1.10	0.88	0.56	0.20
$3\frac{1}{16} \times 2\frac{3}{4} \times \frac{5}{16}$	1.88	2.38	1.40	1.11	0.72	0.23
$3 \times 2\frac{11}{16} \times \frac{3}{8}$	2.04	2.57	1.57	1.22	0.81	0.22
$3\frac{1}{16} \times 2\frac{3}{4} \times \frac{7}{16}$	2.38	2.98	1.88	1.44	0.98	0.24
$3 \times 2\frac{11}{16} \times \frac{1}{2}$	2.45	3.06	1.99	1.51	1.05	0.22
$3\frac{1}{16} \times 2\frac{3}{4} \times \frac{9}{16}$	2.76	3.43	2.30	1.71	1.22	0.23
$4 \times 3\frac{1}{16} \times \frac{1}{4}$	2.02	3.14	1.44	1.37	0.74	0.42
$4\frac{1}{16} \times 3\frac{1}{8} \times \frac{5}{16}$	2.54	3.91	1.84	1.74	0.95	0.48
$4\frac{1}{8} \times 3\frac{3}{16} \times \frac{3}{8}$	3.06	4.67	2.26	2.10	1.18	0.53
$4 \times 3\frac{1}{16} \times \frac{7}{16}$	3.13	4.83	2.37	2.20	1.25	0.50
$4\frac{1}{16} \times 3\frac{1}{8} \times \frac{1}{2}$	3.60	5.50	2.77	2.54	1.47	0.53
$4\frac{1}{8} \times 3\frac{3}{16} \times \frac{9}{16}$	4.06	6.18	3.19	2.88	1.71	0.56
$4 \times 3\frac{1}{16} \times \frac{5}{8}$	3.94	6.05	3.18	2.86	1.73	0.50
$4\frac{1}{16} \times 3\frac{1}{8} \times \frac{11}{16}$	4.35	6.65	3.58	3.18	1.97	0.51
$4\frac{1}{8} \times 3\frac{3}{16} \times \frac{3}{4}$	4.77	7.26	4.00	3.50	2.22	0.52
$5 \times 3\frac{1}{4} \times \frac{5}{16}$	4.80	5.34	2.00	2.26	1.04	0.80
$5\frac{1}{16} \times 3\frac{5}{16} \times \frac{3}{8}$	3.38	6.39	2.45	2.74	1.29	0.90
$5\frac{1}{8} \times 3\frac{3}{8} \times \frac{7}{16}$	3.97	7.44	2.92	3.22	1.55	0.98
$5 \times 3\frac{1}{4} \times \frac{1}{2}$	4.02	7.68	3.02	3.35	1.63	0.94
$5\frac{1}{16} \times 3\frac{5}{16} \times \frac{9}{16}$	4.55	8.62	3.47	3.80	1.90	0.99
$5\frac{1}{8} \times 3\frac{3}{8} \times \frac{5}{8}$	5.09	9.57	3.94	4.26	2.17	1.03
$5 \times 3\frac{1}{4} \times \frac{11}{16}$	4.94	9.47	3.91	4.25	2.20	0.94
$5\frac{1}{16} \times 3\frac{5}{16} \times \frac{3}{4}$	5.42	10.34	4.37	4.67	2.48	0.97
$5\frac{1}{8} \times 3\frac{3}{8} \times \frac{13}{16}$	5.91	11.20	4.84	5.10	2.78	0.99
$6 \times 3\frac{1}{2} \times \frac{3}{8}$	3.85	8.44	2.75	3.48	1.46	1.36
$6\frac{1}{16} \times 3\frac{9}{16} \times \frac{7}{16}$	4.52	9.83	3.27	4.10	1.75	1.50
$6\frac{1}{8} \times 3\frac{5}{8} \times \frac{1}{2}$	5.20	11.22	3.81	4.72	2.06	1.62
$6 \times 3\frac{1}{2} \times \frac{9}{16}$	5.24	11.55	3.91	4.88	2.15	1.57
$6\frac{1}{16} \times 3\frac{9}{16} \times \frac{5}{8}$	5.87	11.82	4.44	5.47	2.47	1.65
$6\frac{1}{8} \times 3\frac{5}{8} \times \frac{11}{16}$	6.50	14.10	4.98	6.07	2.80	1.71
$6 \times 3\frac{1}{2} \times \frac{3}{4}$	6.32	14.04	4.94	6.06	2.83	1.60
$6\frac{1}{16} \times 3\frac{9}{16} \times \frac{13}{16}$	6.89	15.22	5.47	6.62	3.17	1.64
$9\frac{1}{8} \times 3\frac{5}{8} \times \frac{1}{7}$	7.48	16.40	6.02	7.18	3.52	1.67



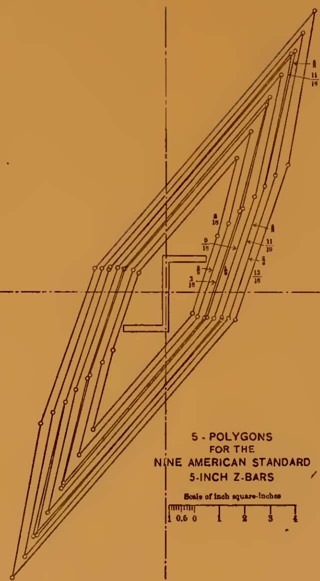
S - POLYGONS
FOR THE
SIX AMERICAN STANDARD
3-INCH Z-BARS

Scale of inch square-inches



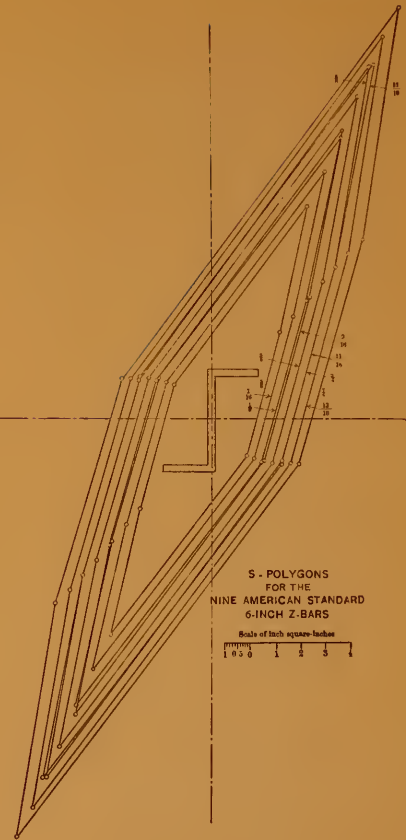
S - POLYGONS
FOR THE
NINE AMERICAN STANDARD
4-INCH Z-BARS

Scale of inch square-inches



S - POLYGONS
FOR THE
NINE AMERICAN STANDARD
5-INCH Z-BARS

Scale of inch square-inches



S - POLYGONS
FOR THE
NINE AMERICAN STANDARD
6-INCH Z-BARS

Scale of inch square-inches

APPENDIX.

11. The Equation and Properties of the Neutral Axis.—Setting $f = 0$ in Equation 10, and rearranging, the equation of the neutral axis is found to be

$$y = \frac{x_k I_x - y_k J}{x_k J - y_k I_y} x + \frac{I_x I_y - J^2}{A(x_k J - y_k I_y)} \dots\dots\dots\dots (17)$$

the coefficient of x being its slope, and the constant term its intercept, b, on the axis of Y. These values might also have been obtained as tan. α. from Equation 7, and v_o sec. α, from Equations 5 and 6.

Inspection of Equation 17 will reveal a number of important facts.

In the extreme case, when K is at G, y_k and x_k vanish, tan. α becomes indeterminate, and b becomes infinite. That is, if N be applied at the center of gravity of a section, the neutral axis will be at an infinite distance, and with no determinate direction. The f is then constant throughout the section, as might have been foreseen. This is the case of pure normal stress (compression or tension).

For the other extreme, where K is at infinity (the condition for pure flexure), either or both of x_k and y_k become infinite, tan. α is determinate in any given problem (Equation 7) and b vanishes. That is, in pure flexure the neutral axis passes through the center of gravity of the section at an inclination depending upon λ.

For intermediate positions of K, the neutral axis has a determinate slope, and is at a finite distance from G, dependent upon the location of K, and the greater the distance, $K\ G$, the nearer to G will be the neutral axis, and *vice versa*.

It can be shown that the quantity, $I_x I_y - J^2$ is never negative With the aid of this fact, and by finding the intercept of a line through K parallel to $n\ n$, it will be seen that K and $n\ n$ always lie upon opposite sides of $G\ U$, as might have been foreseen.

Since tan. α depends solely upon the ratio of y_k and x_k, and not at all upon their actual magnitudes, it is evident that, for all K's on a line through G, the neutral axes will be parallel, and conversely.

Further, the intersection of neutral axes for any two K's upon any straight line, $y = l\ x + b$ will be found to have for its coordinates

$$\left. \begin{aligned} x &= \frac{J - l\ I_y}{A\ b} \\ y &= \frac{I_x - l\ J}{A\ b} \end{aligned} \right\} \dots\dots\dots\dots\dots\dots (18)$$

quantities dependent only upon the constants of the given line and of the section. Hence it appears that for K's moving on any straight

line the neutral axis rotates about a fixed point, given by Equation 18, and conversely. If the line passes through G, b vanishes, x and y become infinite, and the statement of the preceding paragraph is confirmed.

12. The Kernel of a Section.—The kernel of a section is the area bounded by the locus of the K's corresponding to a series of neutral axes touching the periphery of the section but never crossing the section. It could be located with the help of Equations 7 and 18 without difficulty. Its main interest lies in its defining an area within which a K must fall in order that the unit-stress may be of the same sign throughout the section. It is interesting, too, in that its radii vectores are lengths which, for any λ, need only to be multiplied by the area of the section to give S' and S'' for that λ. These lengths would have to be called $+$ if on the opposite side of K from G, and *vice versa*. The kernel is then a figure with sides respectively parallel to the S-polygon, but on opposite sides of G, but unlike the S-polygon in being geometrically an actual surface and to the same scale as the section.

The kernel will be established if the K's be found for the neutral axes coinciding with the sides of the circumscribing polygon of the figure. For each such side considered as a neutral axis there will be a K which will be an apex of the kernel. In the light of the preceding paragraph, the co-ordinates of the kernel apex corresponding to any such side, AB, may be easily written by reversing the signs in Equation 16 and by dividing the second member by A. Accordingly,

$$\left.\begin{aligned} x_{ab} &= -\frac{(x_a - x_b)J - (y_a - y_b)I_y}{A(x_a y_b - x_b y_a)} \\ y_{ab} &= -\frac{(x_a - x_b)I_x - (y_a - y_b)J}{A(x_a y_b - x_b y_a)} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (19)$$

are co-ordinates of a kernel apex corresponding to AB. If AB fall out parallel to the X-axis, these expressions become

$$\left.\begin{aligned} x_{ab} &= -\frac{J}{A\,y_a} \\ y_{ab} &= -\frac{I_x}{A\,y_a} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (19')$$

and, if parallel to the Y-axis, they become,

$$\left.\begin{aligned} x_{ab} &= -\frac{I_y}{A\,x_a} \\ y_{ab} &= -\frac{J}{A\,x_a} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (19'')$$

expressions paralleling Equations 16′ and 16″.

The kernel and the circumscribing polygon of the section are related to each other in such a way that if the K travel along the

one, the corresponding neutral axes will roll around the other, meaning, by that, that they will coincide with side after side of the other polygon, and, pivoting about the apices of the other polygon, will assume all intermediate positions possible without crossing the surface of the polygon.

The continental writers have given much attention to the kernel, but its practical usefulness is so slight, compared with its close kin, the S-polygon, that further description of it need not be given here. The preceding section is here given primarily in preparation for the following section.

13.—The S-Polygon and W-Fläche Compared.—The S-polygon arrived at in the foregoing was the result of an independent attempt to establish the *W*-Fläche of Professor R. Land, of Constantinople. The only difference between the two lies in the fact that the sides of the two polygons for a given m lie on opposite sides of G, making the *W*-Fläche just like the kernel with all its dimensions multiplied by A. The writer believes that the maintenance of this distinction, even at the expense of losing the closest possible similarity with the kernel, is worth while. The relations with the kernel will be of little importance in practice, and it seems natural, and hence conducive to accuracy in computation, to call the radius vector positive rather than negative when measured on the same side of G as the point the stress of which it determines. The radius vector will then come to be a kind of pointer starting from the natural origin, G, toward the portion of the cross-section to which it belongs. This is a valuable safeguard against confusion, especially as the diagram is one the use of which is within the comprehension of computers of very moderate experience. Finally, for all sections symmetrical about G, such as I's and Z's, the *W*-Fläche and S-polygon would be indistinguishable except in the lettering. Only for sections like [’s, and T's would the difference appear.

14.—Closing Remarks on the S-Polygon.—The S-polygon is a graphic exhibit of the values of S leading to the extreme unit stresses in the section for all possible inclinations of the plane of loads. It is subject to the advantages and disadvantages of graphical work. It is of the greatest possible simplicity in use, but the measurement of the radii vectores to a high degree of precision may be difficult. A degree of precision quite sufficient for structural designing, however, is easily obtained. Extreme accuracy is rarely justified in practice, considering among other things the uncertainty as to the actual magnitude and inclination of the loads, the inevitable variations of the rolled shape from the nominal section, inaccuracies in construction and erection, and the doubt whether the principle of linear distribution of stress is rigorously applicable. Such considerations make it certainly justifiable to ignore the

minute effect of the rounded fillets and corners of the familiar shapes.

If, for any reason, the computed value of S be required, it can easily be found by substituting the proper x, y, and λ in Equation 11. In fact, a numerical table could thus be computed for as small variations in λ as desired, which would parallel the graphic S-polygon. Such tables have been published in Germany for some of the German standard rolled sections.

Finally, the S-polygon should be observed to be bounded by lines the radii vectores of which for all values of λ have a significance. Radii vectores for points in these lines produced measure values for S which, upon insertion in Equations 13 and 14 will give correct values for the particular point, m, to which the S-line belongs, and the particular λ. The portions of an S-line between two successive apices of the S-polygon are of special importance simply because they mark out the limits of λ's for which an extreme fiber stress will result from the corresponding low values of S.

In closing, the writer wishes to express his obligations to Bruce Borland, Jun. Am. Soc. C. E., who performed most efficiently the exacting task of computing Table 2; and Messrs. H. E. Norton, S. B., J. R. Nichols, and H. W. Telford, S. B., who have assisted in various ways in the preparation of this paper.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THE CONTROL OF HYDRAULIC MINING IN CALIFORNIA BY THE FEDERAL GOVERNMENT.

By William W. Harts, M. Am. Soc. C. E.*

To be Presented May 2d, 1906.

As far back in the past as history takes us, the search for gold has had a passionate interest for mankind. The same zeal that animated the Argonauts who accompanied Jason in his quest of the Golden Fleece, the same patience and perseverance that urged on the searchers for the philosopher's stone, the insatiable greed for quickly acquired wealth that sent the Spanish conquerors to the New World, are even in this day driving men to deeds of daring and adventure at the farthest ends of the earth.

The richness of the treasure discovered in California a generation or more ago seems almost fabulous. Gold fields that yielded in a single year more than $81 000 000, and since the first discoveries have produced up to date more than $1 414 000 000, have filled the minds of an army of miners, from that day to this, with that fever for gold which, once acquired, seems never to relax its hold on its victim.

* Captain, Corps of Engineers, U. S. Army.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Scarcely a stream of the western slope of the Sierras but held in its gravel bed quantities of this precious metal, and when all these had been exhausted it was found that enormous wealth could be extracted from the old gravel beds of the prehistoric rivers, those of the tertiary age. To wash through sluice boxes in the quickest time possible these vast quantities of gravel, in order to secure the gold contained, was the eager effort of thousands of men. Millions of cubic yards of these old gravel deposits were thus displaced every year, and washed into the streams and watercourses.

The State of California, not realizing or not caring that some day this might lead to much trouble, assisted the miners in every conceivable way. The right of condemnation of private property for rights of way for mining ditches was granted to the miners, the use of the streams to carry away the tailings of the mines was at one time authorized by legislation, and even the Federal Government, on whose public lands these mines were mainly located, not only made no charge for the privilege of extracting the gold, but even authorized the miners to take up claims without charge, in which they could have a temporary possessory right, protected by law as long as they worked their ground.

It is no small wonder that little thought was given to injuries which might be caused by these operations.

The topography of the Sacramento and San Joaquin Valleys is peculiar, and any interference with the outflow of the floods of these rivers is certain to aggravate a condition already a very troublesome one by nature. Along the flanks of these rivers for miles are rich areas, sometimes several miles wide, which are overflowed each year because of the inability of the rivers to carry off the floods caused by the spring rains and melting snows. For months these rich lands are under water. As the population of the valleys along the rivers increased, a growing hostility arose toward the miners who, it was believed, were mainly responsible for these conditions. Thus arose the historic contention between the miners on the one hand and the valley people on the other, which led to much bitterness of feeling in later years. Its settlement has occupied the minds of many engineers and legislators for years, as well as the time of many lawyers and judges. The end is only now barely in sight.

The Debris Problem.—The problem of what to do with the debris

resulting from years of unrestricted hydraulic mining has thus vexed the people of California for upwards of a generation. In the fierce and bitter conflict between the mining interests of the mountains on the one hand, and the interests of the agricultural regions on the other, which has been carried on for more than thirty years, the industry of hydraulic mining was finally completely vanquished. The decisions of the Courts, and their injunctions, had put an end by about 1880 to a form of mining activity in which more than $100 000 000 were said to have been invested at that time, and from which the State of California had originally derived most of its claim to the attention of the world. These decisions and injunctions against hydraulic mining were based on the injuries caused by the tremendous inundation of mining detritus over the farming lands and river beds of the valleys of the central part of the State, and on the constitutional right of the owners of this property to be protected from such injuries.

It cannot be denied, however, that the discovery of gold in California in 1848 gave a stronger impetus to the growth of the State than any other single occurrence before or since. By 1849 there were 100 000 miners engaged in washing the gravel of the river beds of the Sierras. Where a sparse and scattered population had worked out a more or less precarious existence, the discovery of gold caused mining camps first and then flourishing towns to spring up as if by magic. As these grew, the mining industry was followed by the farmers, orchardists and ranchers who settled later in the rich valleys of the Sacramento and San Joaquin Rivers. To save their farms from inundation caused by raising the flood plane of the rivers, and to prevent the bottom lands from being ruined by the accretion of gravel and debris from the mines in the mountains and foothills, legal remedies were resorted to by the valley people, and by about 1880 the once enormous industry of hydraulic mining had been successfully throttled by the injunctions of the Local and Federal Courts.

The feeling between the miners and the valley people became acute, and much bitterness resulted. The State of California had extensive investigations made, but found it could not cope with the difficulty. The Federal Government was then appealed to, and examinations were ordered by Congress in 1880 and again in 1888.

The former was made and a report submitted by Colonel G. H. Mendell,* Corps of Engineers, U. S. Army. The latter was made by a board of three officers consisting of Lieutenant-Colonel Benyaurd, Major Handbury and Major Hauer, all of the Corps of Engineers.† On the report of this board, Congress passed in March, 1893, a mining law which has been since known as the Caminetti Act, from the name of its proposer in Congress. This Act was ostensibly for the protection of the navigability of the Sacramento and San Joaquin Rivers, by preventing them from being filled with mining debris and their depths thus reduced to a point where free navigation would be impossible. It was fervently hoped by those interested in mining in California that the application of the law would be far wider in its benefits and give them the long-sought privilege of operating their mines without hindrance. This law created a Federal commission to regulate hydraulic mining in such a way as to prevent any injury to navigable waters, with ample power to stop all such mining until satisfactory impounding facilities for its detritus were provided by each mine.

The constitutionality of such an enormous interference with the interior affairs of one of the States of the Union as was embraced in this law was based on the provision of the Constitution of the United States assigning to the Federal Government the duty of regulating the commerce between States, and incidentally of protecting the navigable highways.

It was presumably understood in Congress that the steps necessary to protect the interests of navigation from the mining debris evil would necessarily protect all the agricultural interests, as the greater includes the less, so that if the engineers in charge could devise a method which would protect satisfactorily the navigability of the rivers affected, the other questions involving private rights would solve themselves. The depths of the rivers being affected so quickly, there was thus afforded a delicate test of injury. Any mining that would cause no appreciable shoaling in the navigable rivers could scarcely be of any injury to any other interests. If the navigability of the rivers was protected, therefore, the problem that had been of such long standing could be soon settled. This law, it was

* Ex. Doc. 76, House, 46th Cong., 3d Session.

† Ex. Doc. 267, House Reps., 51st Cong., 2d Session.

thought, would thus permit the resumption of hydraulic mining under such restrictions as might be necessary to protect the rivers.

The Act provides for a Federal board of three engineer officers of the army, appointed by the President and confirmed by the Senate, to be called the California Debris Commission. It became their duty under the law to devise plans that would permit hydraulic mining to be resumed, under sufficient restrictions to prevent injury to the lower rivers and the land lying adjacent thereto. The Commission's jurisdiction was limited to the water-shed of the Sacramento and San Joaquin Rivers, but within these limits all hydraulic mining was prohibited except with the license or permit of this Commission. A maximum penalty of $5 000 fine and one year's imprisonment was imposed for violation of the Act.

The Commission was directed also to devise projects that would improve the navigability of the rivers above named and their tributaries, protect their banks against encroachment and damage from mining debris, and restore, as far as required by the needs of navigation, the navigability of the rivers as it existed in 1860.

What has been accomplished by this Commission since its organization is the subject of this paper.

Description of the Mining Region.—To understand fully what was assigned to the California Debris Commission and the wide scope and difficulty of its work, the geography and topography of the State of California must be considered briefly in connection with the flow of mining debris.

The entire central portion of the State is inclosed by mountains and consists of an elliptically shaped valley about 450 miles long and 40 miles wide, embracing about 18 000 sq. miles. It is hemmed in by the Coast Range on the west and the Sierras on the east, and is drained by the Sacramento and San Joaquin Rivers, which empty into the Pacific Ocean through San Francisco Bay. All the water falling on the areas of the water-shed of these rivers must pass into the Pacific Ocean through one mouth, about a mile wide at its narrowest point, called the Golden Gate.

The flat portion of the valley embraces about 4 769 sq. miles, where the annual rainfall is about 18 to 20 in.; the Sierra slope, with elevations as high as 11 000 ft., has an area of about 8 843 sq. miles, where the average rainfall varies from 24 to 102 in.; the slope of the Coast

Range has an area of 3 075 sq. miles, with a rainfall of something less than the Sierra slope, whereas the Shasta region on the north, having elevations of more than 14 000 ft. and an area of 5 616 sq. miles, has a rainfall ranging from 30 to 110 in. The average rainfall over the entire water-shed of the Sacramento Basin, of about 26 187 sq. miles, may be averaged at about 30 in.*

Practically all this rainfall is during the winter months and in the early spring, so that the warm rains and melting snows together frequently cause excessive floods. The watercourses in their upper portions are all mountain torrents. It thus happens that every spring the Sacramento River is required to carry off floods far beyond the capacity of its bed.

The superficial area of the channel and slough surface is only 38 sq. miles, out of a total of 4 769 sq. miles of the level part of the valley, and the fall in surface slope is very limited in the lower portions of the river. It has been estimated that, with an annual precipitation of from 30 to 40 in., there will be an area of about 1 700 sq. miles, more or less, flooded every spring when a warm rain sets in after heavy snows.†

It is very evident that the local conditions have for ages, if not always, prevented the Sacramento River from discharging its flood waters promptly. Furthermore, for ages the well-known erosive action and leveling tendencies of all rivers have been at work cutting down the mountains and hillsides and filling up the lower river reaches and the bays. Much of the Sacramento Valley has unquestionably been formed in this way, and this action is doubtless still in progress. It seems more than likely that Suisun Bay is all that is left of a large inland sea, the waters of which at one time probably washed the lower slopes of the Sierras.

The denudation of various valleys by natural causes in different parts of the world progresses at different rates, and it is unfortunate that no definite records are available for the Sacramento water-shed.

Observations on the quantity of sediment carried by various rivers in various lands indicate that the time required by natural agencies to remove an average thickness of 1 ft. of rock in each of the several drainage basins is as follows:

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* "Physical Data and Statistics of California," by William Ham. Hall, M. Am. Soc. C. E.

† *Transactions*, Technical Society of the Pacific Coast, February and March, 1887.

Danube Basin	6 846	years.
Mississippi "	6 000	"
Nile "	4 723	"
Ganges "	2 358	"
Rhone "	1 528	"
Hoang Ho "	1 464	"
Po "	729	"

Applying the Mississippi rate to the Sacramento River, it would indicate a removal of about 5 600 000 cu. yd. of sediment per annum; at the Po rate it would indicate 43 000 000 cu. yd. The natural conditions of the Sacramento Basin resemble those of the Po more than those of the Mississippi. It is stated as probable that the annual sediment moved by natural causes is somewhere between the rates of these two rivers, that is, between 5 600 000 and 43 000 000 cu. yd. per annum for the Sacramento Basin.*

Nearly all this natural wash is carried into the lower sections of the tributary rivers, some to be transported into the Pacific Ocean with the tidal currents, some to settle on the shoals and tide flats around San Francisco Bay and some to add to the already gorged condition of the river channels.

We thus see that for probably thousands of years natural agencies have been at work filling up the navigable rivers and encroaching on the area of San Francisco Bay. These natural agencies, already promising certain disaster to the navigable capacity of San Francisco Harbor at some far-distant day, were assisted at an enormous rate by the work of Man, for there were recently added human agencies that rendered more complicated an already too complex situation. This was the removal by hydraulic mining of millions of cubic yards of gravel and sand from their location on the western slopes of the Sierras and the washing of this material into the cañons and tributaries of the Sacramento, Feather, American and San Joaquin Rivers.

Hydraulic Mining Methods.—Gold was discovered in California in 1848 during the construction of a mill race at Coloma, near Georgetown, purely by accident. The news spread rapidly, and in 1849 there followed an army of 100 000 miners, who soon washed out

* Annual Report, Chief of Engineers, U. S. Army, 1882.

all the accessible gravel lying in the beds of the small creeks and rivers.

The hand-pan, in which only a small quantity of gravel at a time could be washed, gave way early to rockers of larger capacity, and these to sluices where riffles caught the gold, after being washed and separated by the action of flowing water, brought in ditches, sometimes for great distances. It was only a step further to the use of hose and nozzles, and then to steel pipes and monitors for directing powerful streams against the tertiary gravel banks of higher elevations. The streams are used under pressure to break down the banks, and the water carries off the gravel through the sluice boxes, where the gold, by virtue of its high specific gravity, is caught in riffles of various kinds.

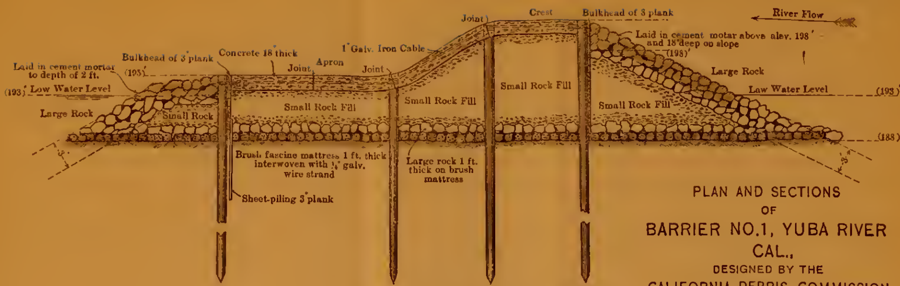
This method was invented by a native of Connecticut, and dates from 1852. It dispenses with many men and is a cheap and quick method of excavating gravel banks. The cost, in favorable situations, has been stated to have been as low as 3 to 6 cents per cu. yd. Nozzles as large as 9 in. in diameter have been used, pressures as high as that due to 400 ft. head, and quantities as great as 3 500 miners' in. (about 87 cu. ft. per sec.) in a single stream. The excavating power of these large streams is enormous.

The engineering features connected with these early mining operations are full of interest even now. The difficulties were tremendous in building mining ditches carrying several thousand inches of water for sometimes 40 and 50 miles along the crests of ridges and over deep gulches in wooden flumes, all to bring the quantity of water necessary to mine the gold, at an elevation sufficient to give the pressures that were indispensable. In securing outfall at the mines sufficient to remove the debris while mining, tunnels of great length, in rock, were also often necessary. These operations were all on a scale but little realized by those who have not visited the mining regions.

Extent of the Damage.—The quantity of gravel which can be moved in 24 hours by each miners' inch of a stream is called its "duty." This depends on the character of the bank, and varies from ½ cu. yd. to 6 cu. yd., with an average of about 3 cu. yd. This duty, with an approximate knowledge of the total quantity of water used in a season throughout the entire mining field, gives some

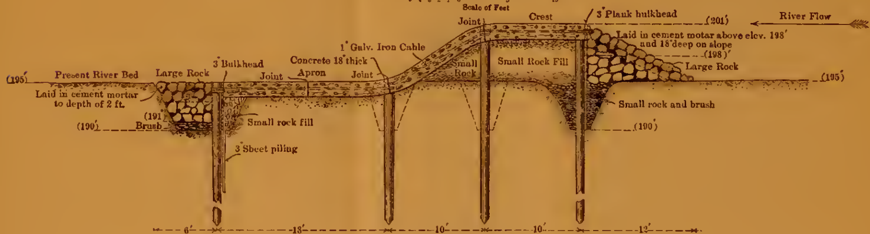
PLATE VI.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1906.
HARTS ON
CONTROL OF HYDRAULIC MINING.

PLAN AND SECTIONS OF BARRIER NO.1, YUBA RIVER CAL., DESIGNED BY THE CALIFORNIA DEBRIS COMMISSION



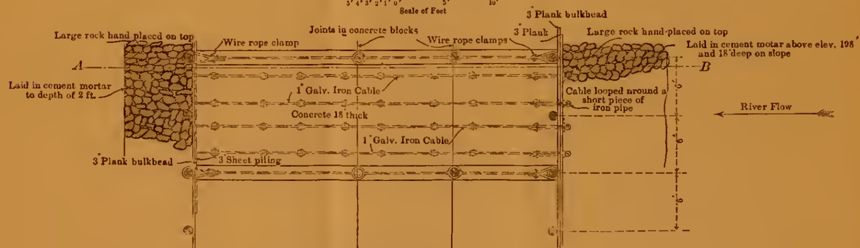
SECTION OF DAM IN CHANNEL

Scale of Feet



SECTION OF DAM ON A B.

Scale of Feet



TOP PLAN OF DAM

Scale of Feet

idea of the quantity of material dislodged annually. In this way it was estimated in 1880 by the State engineers that 53 404 000 cu. yd. of gravel were washed from their banks into the cañons and rivers of the Sacramento Basin. Much of this material was coarse, and was not carried far, but most of the fine material was unquestionably taken down by the first freshet, to be added to year after year, and carried farther at each high water.

Sediment observations of the water of the Sacramento River were made in 1880, and deductions from them indicated that 18 100 000 cu. yd. were brought down that river in suspension that year, of which amount 4 900 000 cu. yd. were assumed to be the proportion due to natural wash and 13 200 000 cu. yd. that due to hydraulic mining. Of the total, 2 100 000 cu. yd. were estimated to have settled in the overflowed basins along the river, leaving 16 000 000 cu. yd. that were carried in suspension past Sacramento and down into Suisun and San Francisco Bays. Of this amount (16 000 000 cu. yd.), it is assumed that 11 100 000 cu. yd. is the proportion due to hydraulic mining. Deducting this 11 100 000 cu. yd. from the total amount excavated by the hydraulic process (53 404 000), as ascertained by the "duty" of hydraulic streams and referred to above, we find that the remainder, 42 304 000 cu. yd. must have been left behind in the numerous cañons and tributaries of the Sacramento and San Joaquin Rivers as a result of the mining operations of a single year.*

These conclusions are based on what now appears to be insufficient observation, and the reference to them here is mainly intended to give an indication as to the enormous extent of the debris problem. A visit through the mining regions, for example, from Nevada City to Dutch Flat, would dispel any doubts in anyone's mind that the denudation by this means was enormous.

The damage done to the streams and lower country is as palpable and manifest to anyone who has visited the Yuba, the Bear and American Rivers as is the extent of denudation.

The vast quantity of debris now lying in the river-beds has been distributed along the watercourses in accordance with the well-known laws of water transportation. Much of it was undoubtedly left where new conditions of flood would move it again to a new

* See Report, State Engineer, 1880.

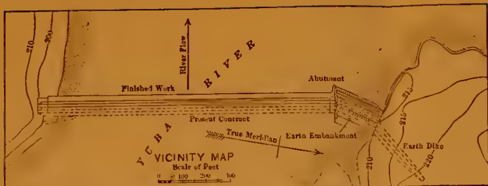
location lower down. It thus seems to be plain that the enormous quantities of tailings in the beds of the cañons and streams are not permanently located, but are only awaiting an opportunity to move to a new location whenever the water velocities of the stream are favorable. The injury once done still exists as a menace for the future, and will continue to exist until conditions of more perfect equilibrium are reached. Surveys recently made indicate that between 1899 and 1904 an addition of more than 15 000 000 cu. yd. was lodged in the bed of the Yuba River from these old deposits.

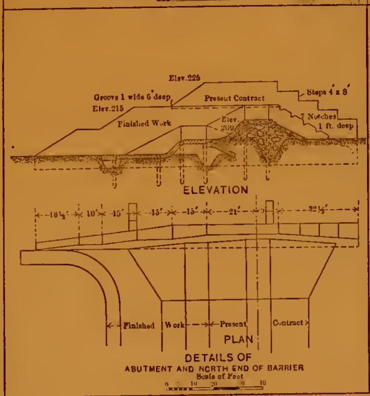
The low-water plane of the Yuba River at Marysville was raised 15 ft. between the years 1849 and 1881.* The citizens of Marysville, in order to protect their town from the damage due to floods occasioned thereby, built levees around the city, which were added to from time to time as required. These levees were continued up to Daguerre Point, about 11 miles, on the north bank. This necessitated levees on the other, or south side, of the river, to protect the adjoining agricultural lands, and thus grew up an extensive levee system which is now maintained at great expense. The lower river, from a narrow mountain stream in 1850, is now filled with sand and gravel so that its bed is nearly 3 miles wide in places, with an average width of about 2 miles. This abnormal width, lying between the lines of levees, extends up the river from Marysville to Daguerre Point, in all about 9 miles. About 25 sq. miles are covered with mining debris. The depth of fill varies from about 7½ ft. at Marysville to 26 ft. at Daguerre Point and 84 ft. at Smartsville. A short distance east from Marysville the bed of the river is now 13 ft. above the level of the surrounding farms. It is plain that any accident to the levees near Marysville would mean a disaster to the town.

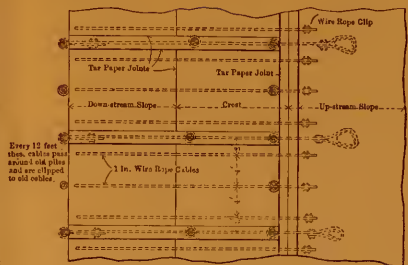
The quantity of material lodged in the river due to mining has been variously estimated at from 71 000 000 to 700 000 000 cu. yd., but it seems safe to say that there are now upwards of 333 000 000 cu. yd. in the bed of the lower Yuba. This material varies in character from cobbles, near Smartsville, to an impalpable powder near Marysville, being distributed along the river-bed from coarse to fine as the slopes diminish. These slopes decrease from about 15 ft. per mile near Smartsville to about 5 ft. per mile at Marysville.

* Appendix MM, Annual Report, Chief of Engineers, U. S. Army, 1882.

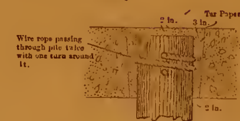
PLAN AND SECTIONS
OF
SECOND STEP
BARRIER NO. 1 YUBA RIVER CAL.
DESIGNED UNDER THE DIRECTION OF THE
CALIFORNIA DEBRIS COMMISSION



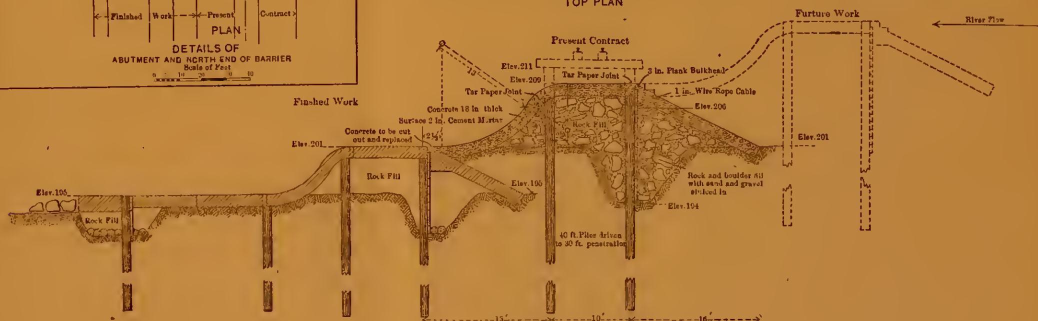




TOP PLAN



DETAIL OF PILE HEAD
SHOWING CABLE FASTENING END JOINTS BETWEEN CONCRETE BLOCKS



SECTION OF NEW YORK
SHOWING CONNECTIONS WITH EXISTING WORK

Not even a faint conception can be had of the vast body of mining material in this river, without seeing the river-bed at low water.

The Bear River shows greater depths of mining detritus than the Yuba, but the quantities are not as large, the cañon of the Bear being steeper and narrower. The best information available leads to the conclusion that the Bear River is filled 150 ft. deep at the crossing between Little York and You Bet. The quantities lodged in the Bear River were roughly estimated at 66 000 000 cu. yd. in 1891.*

The Feather, the American, Consumnes, Calaveras, Mokelumne and other tributaries of the Sacramento are all, in greater or less degree, affected in the same way.

The Sacramento River itself shows unmistakable signs of considerable fill, largely due to mining detritus. Since 1849 the low-water plane at Sacramento has been raised about 7½ ft., causing a reduced carrying capacity for its flood waters and requiring property owners to build levees to protect themselves from floods. The available depth for navigation, however, has remained about the same. The quantity of debris in the river is estimated at 108 000 000 cu. yd.†

A comparison of charts shows that the bar across the entrance to San Francisco Bay has not deteriorated noticeably, nor the channels within the bay diminished in depth, but shoaling on the areas along the flanks of the channels is perceptible, and the channels are slowly narrowing. In Suisun Bay the area of shoals is increasing, and at Carquinez Straits there are indications of the formation of large deposits.‡

Duties of the Commission.—Such, then, was the condition that confronted the first Commission in 1893. To check the injury to navigable streams at once, to prepare plans for the improvement of their navigability by restraining works, and to rehabilitate hydraulic mining as far as practicable, were the duties imposed upon this Commission by the Act. The supply of new detritus from hydraulic mines had by that time been mainly, if not entirely, stopped by injunctions of the Federal and State Courts, prohibiting all such mining, but the flow of debris was still continuing, due to the washing

* Appendix MM, Annual Report, Chief of Engineers, U. S. Army, 1882.
† Ex. Doc. No. 267, House, 51st Cong., 2d Session.
‡ Appendix MM, Annual Report, Chief of Engineers, U. S. Army, 1882.

down at every flood of a part of the enormous quantities already stored in the upper river reaches and the tributary cañons. The damage once started could not be wholly overcome by injunctions.

In designing restraining works there was no precedent nor previous experience from which valuable lessons might be learned by the new Commission, but everything had to be originated *de novo* as no such condition probably exists elcwhere in the world.

In addition to all this, the miners were clamoring for permission to resume mining under the new law, insisting that they be told of the restrictions so that they might comply with the law, if found practicable, and be permitted to resume their business legally.

The first efforts of the Commission were thus directed to making examinations of the mining regions, in order to see what relief could be given the miners.

It was found in many cases that the construction of dams in the cañons below the mines would store all the material that would be moved, and in some cases there were old hydraulic mine pits that could be filled with detritus without damaging the lower rivers. Other expedients were studied, but the method of using dams across the ravines and cañons below the mines was found most satisfactory for the general case.

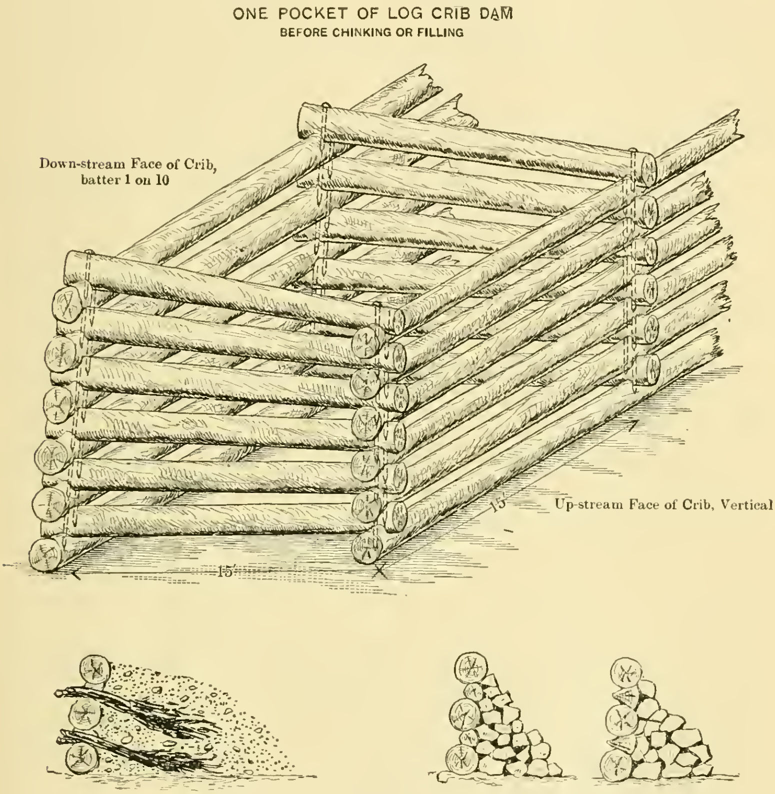
Impounding Dams.—Various kinds of dams were tried: stone, earth, brush and rock, log-crib filled with rock, and many others. After twelve years of experience it has been found that the usual small mine where impounding dams can be used will need one of two general types—either log-crib dams or brush dams.

There are special cases, of course, when other kinds of dams are needed, but these two types are most common for the smaller mines, and printed specifications for these dams with a cut explaining their construction has been prepared by the Commission.

The log-crib is the usual type. It consists of a "cob-house" crib made of large logs which are notched and drift-bolted together. It is filled with quarried rock and chinked against leakage. This type of dam is seldom built more than 40 ft. high, this being the limit of safety placed by the Commission for the usual case. These dams are very satisfactory for their purpose when well made. As long as they are kept wet they are practically permanent, and in those locations where the logs rot, due to being dry part of the time, the rock

being angular and well bedded in gravel will resist erosion long after the logs have failed to bind the dam together.

The brush dam is less used, as it is permitted usually only when the water-flow over the dam is small, or when the river is diverted



SKETCHES SHOWING METHODS OF CHINKING DOWN-STREAM FACE

FIG. 1.

through a spillway at one end, and only when the slope of the cañon above is slight. These brush dams are not permitted more than 20 ft. in total height.

Figs. 1 and 2, with the specifications and the photographs herewith will show plainly their construction in both cases.

General Instructions for Log-Crib Debris Dams.

Approved by the California Debris Commission, January 18th, 1904.

1.—The bottom and sides of the dam are to be founded on bed-rock, and the ends of the timbers set into bed-rock wherever practicable, so as to provide a shoulder against which the dam may rest to resist the pressure of the debris when impounded.

2.—The dam will consist of a down-stream and an up-stream wall of logs connected by cross-logs running up and down stream, the walls of cross-logs to be not more than 16 ft. apart.

3.—All logs are to be as large as practicable, and to be well notched and drift-bolted together at crossings.

4.—The distance between the up-stream and down-stream walls of logs is to be not less than one-half the proposed finished height of the dam, and in no case less than 15 ft.

5.—The up-stream wall is to be vertical, and the down-stream wall is to have a slight slope up stream of about 1 ft. in every 10 ft. in height.

6.—The spaces between the logs in the down-stream wall are to be closed by small logs laid inside the dam, or by brush, as shown in Fig. 1.

7.—The dam is then to be filled with stone and chinked with fine brush, leaves, etc., so that while mining is in progress, it will maintain a pool of water at least 2 ft. deep.

General Instructions for Brush Debris Dams.

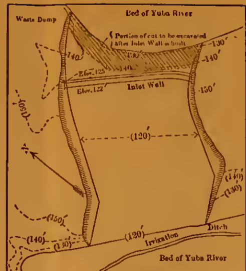
Approved by the California Debris Commission, January 18th, 1904.

1.—Brush dams should be built of live strong brush at least 10 ft. long. All large limbs should be hacked with an axe, but not cut off, and then bent back to lie compactly. Small twigs and leaves should be left on. The poles used should be not less than 4 in. in diameter and not more than 12 in. The poles should be well trimmed, and as long as practicable.

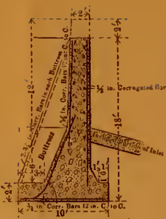
2.—The dam should be built along a straight line, as follows: Level off the foundation. On this lay the brush closely, with the butts in a line and pointed down stream. This should make a thick, compact layer. On top of this layer and at right angles to the brush lay a pole about 2 ft. back from the ends of the butts, which, with other poles like it, should extend entirely across the stream.

3.—A layer of gravel or small stone is then placed on the layer of brush as high as the thickness of the pole. On this layer of

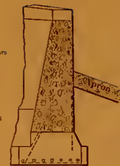
PLANS FOR INLET WALL
AT DAGUERRE POINT CUT,
YUBA RIVER, CALIFORNIA,
FOR CALIFORNIA DEBRIS COMMISSION



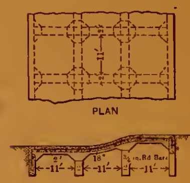
MAP OF DAGUERRE POINT CUT



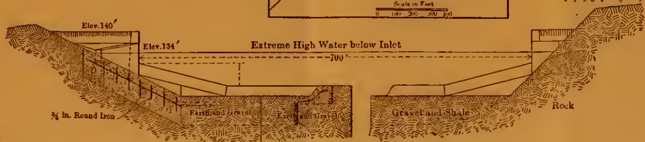
SECTION OF NORTH ABUTMENT
PROBABLE MAXIMUM HEIGHT 16 FT.
DESIGNED FOR HEIGHT OF 20 FT.



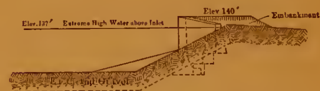
SECTION ON E-F



SPECIAL SECTION OF INLET WALL

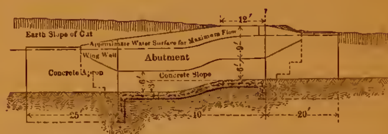


SECTIONAL ELEVATION OF CUT ON LINE A-B-C
LOOKING UP STREAM



ELEVATION OF NORTH ABUTMENT AND INLET WALL
LOOKING DOWN STREAM

PLAN



SECTION OF INLET WALL AND ELEVATION OF NORTH ABUTMENT

gravel place another heavy, compact layer of brush as before, butts down stream and tips up stream, on which lay another row of poles across the stream. Then place another layer of gravel as before, and so continue until the dam is of the required height. The dam should then consist of alternate thick layers of brush and thin layers of gravel, each two layers of brush separated by a row of poles. See Fig. 2.

4.—The poles should be placed so that each row is somewhat back of the row below, so that the whole down-stream face of the dam when completed will have a slope of about 3 horizontal to 4 vertical, and so that the butts of the brush will be about 2 ft. higher than the tips. Each row of poles should be strongly wired, every 4 ft., to the row of poles below.

5.—The dam must be tightened against leakage, with gravel and fine brush thrown on the tips of the brush, so that, when the

BRUSH RESTRAINING DAM
SHOWING METHOD OF CONSTRUCTION

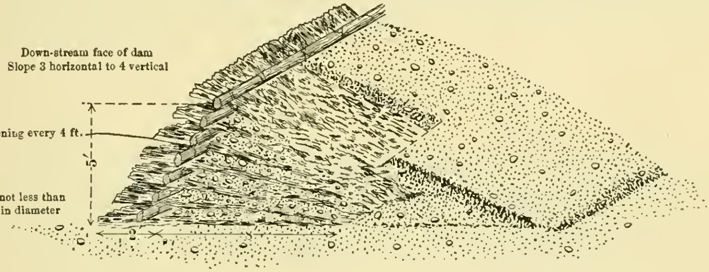


Fig. 2.

mine is being worked, a pool of water at least 2 ft. deep will be always maintained.

Requirements for a License.—Before being permitted to mine, under the United States laws, the hydraulic miner must have a license, or permit, from the California Debris Commission. In order to obtain this license, he must submit an application, or petition, on the form supplied by the Commission, in which he states under oath the location and description of his mine, its extent, the source and quantity of his water supply, the dimensions and grade of his sluice boxes, what restraining works he proposes, the precipitation of rain and snow, the drainage area above his mine, and several other items of information which affect the flow of detritus. This application is advertised in newspapers for three weeks, to permit any protests

to be filed with the Commission. If no protests are received, an inspection of the mine and its impounding facilities is made, and the location and kind of restraining dam decided upon in consultation with the owners of the mine. An order is sent by the Commission to construct the restraining works decided upon, and later another inspection is made when the works are completed. If these works are found satisfactory on this second inspection, a revocable license to mine is issued. Monthly reports are then required from the mines thus licensed, showing the quantity of material mined and the condition of the dams.

Inspections.—Frequent inspections are made to ascertain whether dams are kept in repair, whether there is always ample storage capacity, and whether any mines are operating without licenses or without dams. For this purpose several employees of the Commission, often a Deputy United States Marshal, are constantly inspecting during the mining season, and members of the Commission visit mines from time to time whenever a special inspection is needed. The cost of these inspections, of advertising applications, and the expenses of keeping the records of the mines, their reports, etc., in the office, are all borne by the United States, the mine owners, of course, building and maintaining at their own expense the restraining works required.

The task of inspecting mines and keeping their records is no small one. The mining area covers about 450 miles in length and 40 miles in breadth of rough and mountainous country which must be covered several times each mining season by the inspectors. This is necessary to prevent illegal operations and to maintain control of the licensed mines. The number of applications for licenses now reaches 727, the greater proportion of which, however, are not now in force. A card index, with lists arranged according to counties and consecutive numbers, is necessary to keep accurate record of the conditions at each place.

Mining without licenses, or even dams, and the use of various tricks to avoid building and maintaining dams are practiced to a limited degree, but it is seldom that legal steps are necessary to stop this sort of illegal operation. Careful and frequent inspections, with occasional warnings, are usually all that is necessary to prevent all illegal mining of any importance.

FIG. 1.—BANK OF A HYDRAULIC MINE (NOT IN OPERATION) IN NEVADA COUNTY, SHOWING GREAT DEPTH OF EXCAVATION.

FIG. 2.—DISHONEST SLUICE, WITH CONCEALED TRAP, PARTLY OPEN, TO PERMIT TAILINGS TO PASS INTO A SIDE RAVINE, INSTEAD OF INTO THE AUTHORIZED SETTLING BASIN.

In this way the debris from hydraulic mining has been regulated so that very little is now added to the old supply. These restrictive measures, unfortunately, have been too great to permit the resumption of hydraulic mining on the large scale formerly followed, but, on the other hand, they have permitted many mines to operate that otherwise would have had to remain idle.

There are now mined nearly 1 000 000 cu. yd. each year, which are stored in the cañons and ravines behind debris dams specially constructed for the purpose. Under the restrictions imposed, the lower rivers are now slowly improving. The Sacramento River is gradually lowering its low-water plane at Sacramento, and the effect of the tide is beginning to increase. Both of these indications are distinctly favorable.

Protection of the Navigable Rivers.—The other side of the Commission's duty is the study of the rivers of the Sacramento and San Joaquin systems, with a view to the preparation of plans for the treatment of these streams and their tributaries so that the injurious mining detritus may be kept out of the navigable rivers, and the streams restored to their former condition of navigability as far as may be needed.

The first step, after preventing the operation of mines where debris was not properly impounded, was the treatment of the larger tributaries to prevent the enormous quantities now in their beds from reaching the navigable streams.

In 1881 the State of California built a brush dam in the Yuba and one in the Bear River, with a view to impounding debris up to the crest of the dams. These dams were only a few feet high and were constructed of brush, gravel and sand-bags. They were necessarily founded on the unstable gravel bed of the river. Neither dam withstood the first high water. Both streams have widely varying discharges, the Yuba ranging from about 300 cu. ft. per sec. in the summer and autumn to about 80 000 or 90 000 cu. ft. per sec. during floods. The Bear River varies from about 10 cu. ft. per sec. during the summer low water to an estimated flood discharge of about 15 000 cu. ft. per sec.

With these flood volumes it is evident that any dam to hold, if built on the treacherous gravel of the river bed, must be of considerable strength.

General Principles of Improvement.—In undertaking the formulation of a plan for these rivers, the Engineer Officers adopted a general line of work that was believed to be applicable. It consisted of three divisions:

1.—The construction of moderately high dams in the foothills where the rivers emerge into the valleys, and where the value of land is not great. These dams, being located where the slopes in the river-bed are comparatively high, were placed there with a view to sorting the heavy material, that will stand on high slopes, from the fine material that will not, thus storing this heavy material where it will be impounded cheapest.

2.—Embankments and basins lower down the river, forming settling pools, where the slopes are flatter and where practically all the finer material can be deposited at all except high stages of the river by bringing the flowing water almost, if not entirely, to rest.

3.—Training walls in the remainder of the lower river, to confine the flow in selected channels, so that the large quantities of debris now in the river beds, outside these walls, should not be overflowed, and thus could lie undisturbed indefinitely.

After investigating various other plans, it was believed that the application of these principles would hold back all the debris that could be impounded, and would offer the best solution of the problem.

Yuba River.—It was decided to commence the work of restraining the debris of the Yuba River, as this stream has suffered more from mining detritus than any other in California, and is causing the most trouble in the navigable rivers. If the difficulties could be surmounted in this stream, the methods found best adapted to the purpose would likely be more easily applied to the other rivers.

After a study of several years, and after extended surveys in which numerous borings were made, a plan was adopted by the Commission and submitted to Congress in 1900. Much credit is due to Mr. Hubert Vischer, Assistant Engineer, whose work on this plan, under direction of the Commission, has been of much value. The plan has since been modified from time to time as the progress of the work rendered necessary. The estimated cost was $800 000, of which the State of California, under the provisions of the Caminetti Act, pays one-half. This project was adopted by Congress and funds appropriated therefor.

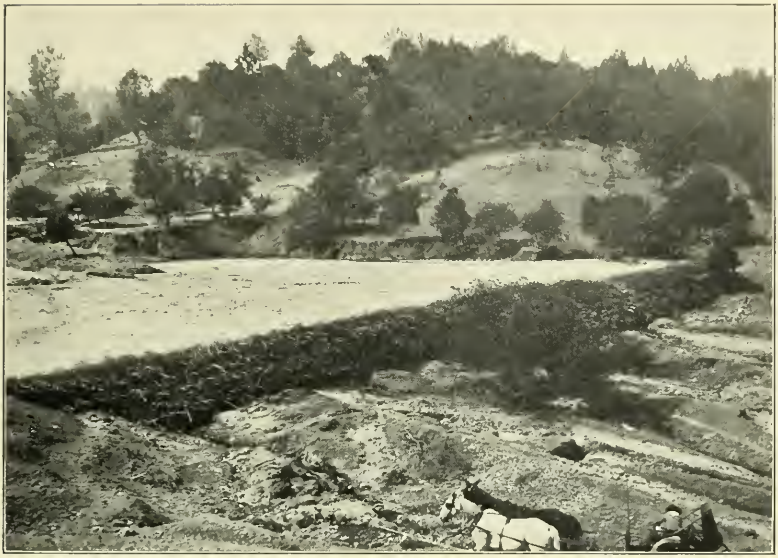


FIG. 1.—A BRUSH DAM IN CALAVERAS COUNTY.

FIG. 2.—A LOG-CRIB DAM, WITH SPILLWAY, IN PLUMAS COUNTY.

The project provided for:

1.—Barriers across the river just below Smartsville, to hold back the coarse detritus coming from the upper reaches.

2.—A cut at Daguerre Point through which to divert the river at high stages, with embankments forming a settling basin for impounding fine material during the remainder of the year.

3.—Training walls, about 2 000 ft. apart, extending from Daguerre Point to the Feather River, to confine the flow to a selected channel.

Barriers.—The barriers are to be a system of weirs extending across the river, where the banks are high enough to afford large

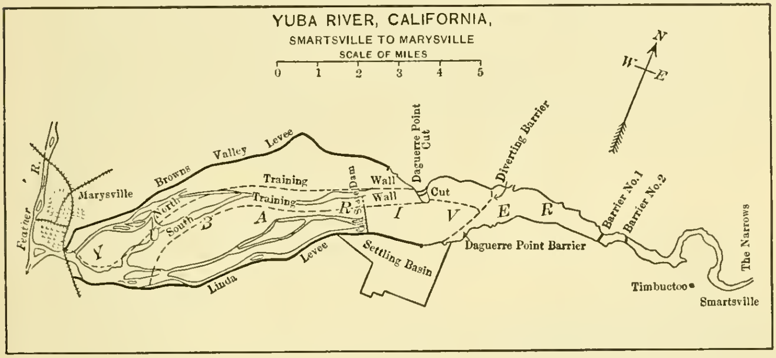


Fig. 3.

impounding capacity, the first located a few miles below Smartsville. This first barrier was the only one estimated for in the present project, but it was to be supplemented by others higher up the river as soon as it was filled, the others to be located and built when necessary.

A dam of brush, rock and gravel was first proposed, with a row of Wakefield sheet-piling 20 ft. deep to protect the toe. It was found impracticable to drive the sheet-piling on account of the coarse and heavy material of the river-bed, and, therefore, this type was abandoned after several hundred feet of it had been placed with much difficulty. This portion afterward washed out during a flood in the winter of 1903-04.

A modified brush barrier was then tried by the Commission. It was a "cob-house" construction of brush fascines forming pens 5 ft.

square with an elevation of 4 ft. above the river-bed. These pens were filled with heavy rock. An apron 20 ft. wide was made of a mattress of brush fascines fastened together with cables. This dam was destroyed by the first high water. The large amount of drift carried by this freshet broke apart the fascines and the dam soon disintegrated.

The design next tried was much stronger. It is anchored to the river-bed with piles, and tied together longitudinally with two timber bulkheads. The first step of this dam, 6 ft. above the river-bed, is comprised of rock fill held in place by concrete blocks weighing about 10 tons each, molded in place over the rock fill, and connected with wire cables embedded in the concrete. Leakage is checked by the timber bulkheads. A broad apron, 20 ft. wide, with a 6-ft. lip, diminishes the scour at the toe, that otherwise might undermine the dam. A sloping up-stream face prevents damage from drift.

It is thus seen that most of the weaknesses of the previous dams have been remedied in the new type. This dam has passed successfully through its first high-water season without any sign of weakness, and is the first dam to withstand a single freshet in the lower Yuba River.

The dam is the first of a series of steps of which the ultimate barrier will be composed and consists of four rows of piles, the two upper intervals between rows being 10 ft. and the interval between the third and fourth rows being 18 ft.

Piles are at 6-ft. centers in the uppermost row, at 12-ft. centers in the two middle rows, and at 3-ft. centers in the lowest row. Every 12 ft. the piles in a tier up and down stream are connected at their upper ends with 1-in. galvanized-wire cable. A timber bulkhead, 3 in. thick, is spiked to the up-stream and another to the down-stream row of piles, and is carried as deep as the water in the river would permit.

Between the first two rows of piles is placed a fill of rock which was brought up to a subgrade, so that when covered by the concrete blocks, 1½ ft. thick, the height of the barrier would be 6 ft. above the average level of the river-bed. Concrete blocks, about 10 ft. square and 1½ ft. thick, are built in place over all this fill, connecting, by a rollerway, with an apron 20 ft. wide resting on the river bed below the dam.

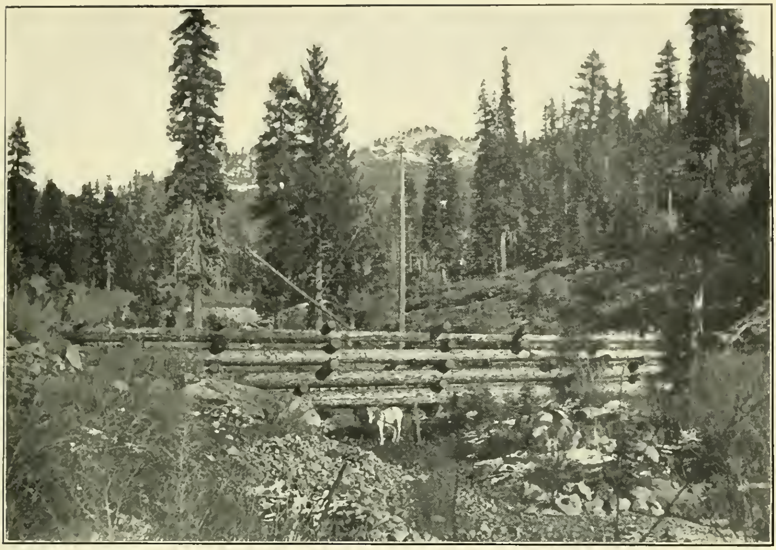


FIG. 1.—A LOG-CRIB DAM IN SIERRA COUNTY, SHOWING LARGE TIMBER USED.

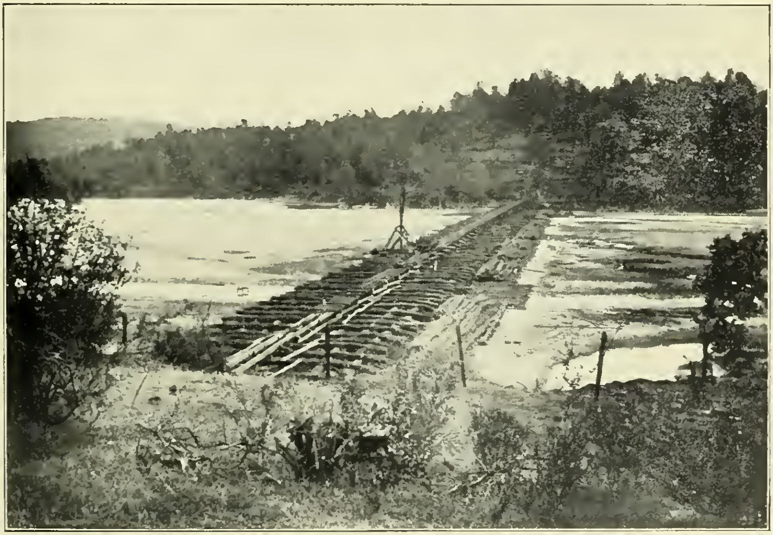


FIG. 2.—A BRUSH AND ROCK DAM ON THE YUBA RIVER, SUBSEQUENTLY DESTROYED BY HIGH WATER.

The up-stream slope is protected by an inclined layer of large rock laid in Portland cement mortar. The concrete slabs of the top surface of the dam and those of the apron are separated from each other by tar-paper joints, and to prevent dislocation by the river currents are connected in the direction of the stream flow with 1-in. galvanized wire cables, 3 ft. apart, embedded in the concrete. The cables connecting the piles referred to above are also embedded in a narrow strip of concrete 18 in. square, which helps bind the heads of the piles together in each tier and separates the large blocks.

Excepting these narrow strips, the concrete slabs rest on the river bottom only, and are not supported on the piles. They are jointed so that they are free to move vertically, the cables acting as hinges.

This was designed to permit the concrete blocks to follow down any considerable scour under the apron, should it occur, and thus prevent any serious damage to the dam due to back-lash.

The weak place in all over-fall dams on poor foundations is, of course, the toe. The rollerway and apron are designed to help to protect the river-bed from excessive and dangerous velocities. In addition, an extension or lip of masonry 6 ft. wide is placed below the apron to carry the water away from the toe, and, if under-scoured, it will break up, fall into the hole and offer protection against further scour. This action occurred near the south bank for a short length during the high water of 1905, and was found to act effectively as planned. In addition, for about 600 ft., the dam at the south end, where the scour was believed to be strongest, was further protected by large rock or rip-rap placed at random. This protection, which may be considered a flexible rip-rap apron, was added to during the next season, making it more than 20 ft. wide entirely across the river-bed.

The south end of the dam is joined to the bed-rock of the river bank, on which the south abutment is founded. At the north end, also, a concrete abutment was built, which was founded on piles, to act as a retaining wall for the earth embankment built later to connect the north end of the dam with the shore. The north shore is composed of compactly cemented gravel through which it was originally planned to have a spillway constructed to carry the river at all stages except flood. During construction, the entire river was

permitted to pass between this abutment and the north shore, before building the embankment, so that the first step was constructed entirely on the dry river-bed. When the time came to close this gap through which the river flowed, it was found to be an undertaking far greater than was anticipated, as there were about 1 200 to 1 300 cu. ft. per sec. flowing around the end of the dam, about three times the flow expected. This flow had to be lifted more than 8 ft. over the completed structure.

The first pile-work placed to close this gap failed on account of the pressure of the water and the serious scour which took place in the bottom while trying to close the opening. Later, more piles were placed in the opening, and, by the liberal use of brush and sand bags, the gap was finally closed.

Much assistance was afforded by an auxiliary dam or levee in the river, ¼ mile long, hastily built to divert the flow over the dam and away from the cut. More than 20 000 sand bags were used in closing this gap, as no rock was available. As soon as closed, the gap was filled with an earth embankment.

The experience with the north bank near the end of the dam has decided the Commission to change the location of the permanent spillway from the north end to the south bank, where the cut will be in rock. This spillway will be 400 ft. wide on the lip and 4 ft. deep, and will be raised as the dam is raised. When completed, it will take a flow of 13 600 cu. ft. per sec. before the dam comes into use, so that the river will be carried around the south end of the dam at all times except for a short period during each year at flood stages. This will make the dam when completed practically permanent and easily maintained.

The first step of the permanent dam having proved so satisfactory, the Commission decided to build the second step during the following low-water season, and contracts were let in May, 1905. This work is now completed. The elevation of the second step is 8 ft., making the total height of the dam 14 ft. The general features of this second step are the same as those of the first step.

In the same way, it is expected to put a step about 8 ft. high on the dam as often as conditions warrant. In this way the ultimate height of the dam (36 ft.) will be reached by successive steps. This method was found advisable, as the amount of work possible in the

FIG. 1.—BARRIER NO. 1, YUBA RIVER, SHOWING METHOD OF CONSTRUCTION.

FIG. 2.—FIRST STEP, BARRIER NO. 1, YUBA RIVER, COMPLETED, READY TO TURN THE RIVER OVER THE DAM.

river-bed during the low-water season is limited, and time must be given to the river to fill each step with gravel.

The first step, already built, has been filled with gravel to its crest since the first heavy freshet, and gravel as large as pigeons' eggs has been rolled over the top of the dam for several months. This has given rise to considerable wear of the concrete, in some places more than 2 in. deep during the winter.

It is thus plainly seen that the dam as a whole is a gravel-fill dam sluiced into place by the river itself, the down-stream slope of which is composed of a layer of concrete blocks having a general inclination of about 1 vertical to 3½ horizontal. The concrete overlies a rock fill held in place by a framework of anchor piles and timber bulkheads.

The concrete for this work was made of gravel taken from the river-bed. It was screened and measured in order to get the proportions of the various sizes decided upon. The materials were unusually clean, white quartz sand, gravel and small cobbles. Cobbles were selected that would pass a 2-in. ring and be held by a 1-in. ring, gravel that would pass a 1-in. ring and be held by a ¼-in. ring, and sand passing a ¼-in. opening was used, care being taken to select sand from places in the bed where but little fine material was found. A number of experiments for voids were made, all showing that the interstices in each size varied from 32 to 35 per cent. The average was about 33%, which was assumed to be the average for each size. The proportions used were: 1 cement, 2 sand, 2 gravel, and 4 small cobbles, for the first season's work, and an additional volume of gravel was added for the second season's contract, making the proportions 1, 2, 3, 4. The concrete was hand-mixed in the first year and machine-mixed in the second year, effecting a great saving in cost to the contractors for the later work. A local brand of Portland cement was used. The resulting concrete was thoroughly satisfactory for the purpose. A wearing surface of 1 to 3 mortar, 2 in. thick, was placed over the concrete in the first year, but this wore through in places with the abrasion caused by the gravel which passed over the dam. It was found that the quartz aggregate of the concrete was the best to resist the abrasion, and the surfacing will probably be omitted in the future.

Cost.—The first piling was purchased under contract, but driven

by the United States, because it was not believed advisable to contract for this work at first, as the risk was considerable, and it was freely predicted by some engineers that no piles could be driven in the bed of the Yuba River at that place. Excepting delay due to overturning and wrecking the driver twice by high water, this work was carried to completion without notable difficulties, 888 piles being driven with an ordinary land driver equipped with a 20-h. p. Lidgerwood engine and a 3 500-lb. hammer working in 45-ft. gins. The driving was difficult, each pile requiring from 150 to 250 blows. The piles cost from 19 to 23 cents per ft. delivered in Marysville, and 10 cents per ft. to haul to the site of the work, 17 miles further. This made each pile cost about $12. The driving cost $5.02 per pile, making the cost of each pile in place about $17. Including experiments with water-jet and accidents, the cost was $19.74. This price was considered high at the time, but the work under the contractor for the piling of the second step, in the spring of 1905, has exceeded this price. Under contract, the new piling for the second step has cost the Commission 31½ cents per ft. per pile delivered at the site of the work, making each pile cost $12.60. The contract price for driving is $7.60, making each pile under contract cost $20.20 in place.

Portland cement costs $2.785 per bbl. delivered at the site of the work. It was furnished by the contractor during the first season, but by the United States during the second.

The cost of the work on the first step, including the abutment, is shown in Table 1, the prices including placing:

TABLE 1.

Excavation	6 470	cu. yd. at	$0.30	$1 941.00
Lumber	61 655	ft., B. M., at	$40.00	2 466.20
Loose brush	95	cords "	3.50	332.50
Large rock	2 411.75	tons "	2.00	4 823.50
Large rock in cement	1 621.307	cu. yd. "	7.00	11 349.15
Small rock fill	4 251	tons "	1.00	4 251.00
Cable	30 300	lin. ft. "	0.18	5 454.00
Concrete	3 754.2	cu. yd. "	7.50	28 156.50
Extra work				1 300.19
Total				$60 073.95

Fig. 1.—First Attempt to Divert River Flow Over the Barrier. Unsuccessful.

Fig. 2.—Portion of River Channel Used During Construction, Closed, Preparatory to Filling with Earth Embankment.

At the prices given in Table 1 only a small profit was made by the contractor, according to memoranda kept by the engineers during the progress of the work.

The entire dam, including piles, but not the north abutment and embankment, cost, in round numbers, \$78 613, or about \$63 per lin. ft.

The construction of the first step of the barrier was successful, and its behavior was satisfactory during the high water of the past winter. This enabled the Commission to take up the construction of the second step of 8 ft. higher during the following low-water season. Accordingly, a contract was let in February, 1905, for delivering 400 piles at the site of the work at 31½ cents per lin. ft., this cost being made up of about 21½ cents on the cars at Marysville and 10 cents for hauling them 17 miles to the barrier. A separate contract was let in February for driving the piles at \$7.60 per pile. All piles, 335 in number, were driven by June 15th, 1905.

The piles were 40 ft. long, driven to 30 ft. penetration in the river-bed, leaving 10 ft. standing for the step, including a small cut-off.

A contract was let for the main part of the second step in April, and by June 15th, 1905, work was commenced. The main features of the second step are similar to those of the first step, and are shown on Plate VII. The work was carried on without interruption from high water, as had happened twice in constructing the first step, and by November 25th, 1905, had been completed. In carrying on this work, the entire river flow, amounting in the late summer to only about 300 cu. ft. per sec., was thrown into a selected channel at the south end of the barrier, which was protected from scour by brush mattress loaded with rock. This enabled the entire dam, of about 1 250 ft. length, except about 120 ft. at the south end, to be constructed in the dry bed of the river. Upon the completion of the main work, a double row of piles at this south end, specially provided for this purpose, was bulkheaded and then filled with rock. This formed a coffer-dam around the unfinished end of the dam, which thus turned the water over the main dam, permitting the small unfinished section to be built without interference.

A flexible apron of rip-rap rock, 20 ft. wide, was laid across the

river-bed below the toe of the first step to protect the bottom from scour. Rocks weighing from 500 lb. to 2 tons were selected for this portion of the dam, 5 472 tons being used. This will be added to, during each low-water season, as needed. A south abutment was commenced, of which the only portion built was that necessary to provide for foundations for future work. This abutment is founded on rock, and will form a part of the spillway to be built later at this end.

The contract prices and the quantities used for this second step are given in Table 2.

TABLE 2.

Excavation........	2 985 cu. yd. at $0.75 per cu. yd.....	$2 238.75
Embankment......	11 520 " " 0.30 " "	3 456.00
Bulkhead lumber...	61 000 ft. B. M. at $25.00 per M.....	1 525.00
Rock fill...........	13 223 tons at $0.875 per ton.........	11 570.13
Large rock........	5 409 " " 2.00 " ".........	10 818.00
Concrete...........	4 156 cu. yd. at $4.25 per cu. yd.....	17 663.00
Wire cable........	15 600 lin. ft " 0.15 " lin. ft.....	2 340.00
Brush mattress.....	560 sq. yd. " 0.70 " sq. yd.....	392.00
Sand bags.........	1 600 at $0.07 each.................	112.00
Total..............		$50 114.88

Records kept during progress of the work show that the contractors made no profits at these figures.

To the total in Table 2 must be added the cost of cement, 4 100 bbl. at $2.785 per bbl., the cost of piles, $6 556, furnished under other contracts, and the cost of superintendence and inspection, amounting to $2 264.

The cost of the second step, not including the abutments or the rip-rap apron below the dam, is $37 575, or $30.06 per lin. ft.

The rip-rap protection below the apron, placed during 1905, consists of 5 472 tons of large rock, costing in all $13 604, or $11.83 per lin. ft.

The cost of the entire work of construction of the first two steps, but not including the end abutments, has been $129 792, or $103.83 per lin. ft.

Daguerre Point Section.—The plans for the treatment of the intermediate section of the Yuba River involve the construction of high embankments of river gravel extending entirely across the

FIG. 1.—PILING COMPLETED FOR SECOND STEP OF BARRIER.

FIG. 2.—SECOND STEP OF BARRIER NEARLY COMPLETED.

river in a **V**-shape, with the apex up stream, the down-stream ends connecting, one with Daguerre Point on the north, and the other with a high knoll on the south bank. These barriers are to be so high that they will never be over-topped.

A diverting barrier connects the apex of the **V** with the north shore, diverting to the south all water below the elevation of its crest. Through Daguerre Point is being cut a channel, 600 ft. wide and 25 ft. deep, through which will pass all the river flow at high stages.

On the south bank, regulating works will admit all the water turned by the diverting barrier, up to a limit of about 3 000 cu. ft. per sec., passing it into a natural depression of about 2 sq. miles area, adjoining the river on the south. These works will exclude all flow greater than what is considered safe, compelling the excess to pass to the north over the diverting barrier and through the cut. This plan is simply taking advantage of the natural regimen of the river.

All rivers have their sections of active erosion, usually where slopes and velocities are highest; their sections of transportation, where slopes and velocities are sufficient to carry sediment, but not to scour; sections of sedimentation, where the reduced velocities permit the sediment to fall and form deposits, and their sections of discharge.

The construction of the embankments and the use of a large settling basin only increase the natural area of the section of sedimentation. The river is passed into an area where the velocities are largely checked, although not entirely overcome, so that practically all their load of sediment is dropped. The water passing back into the river at the lower end of the basin will be practically clear. At stages of the river, occurring for a short time only during each year, when the flow of water through the settling basin would be more than 3 000 cu. ft. per sec. and more than could be settled, the regulating devices will exclude the excess, causing it to pass into the cut. As the river is below this limit for by far the greater part of the year, this will happen only at intervals, and then at such high stages that the velocities are thought to be sufficient to carry whatever fine sediment is in suspension into the Feather and Sacramento Rivers, which, being at flood at the same time, will

carry the sediment into the tidal currents of the bay and thence into the ocean. As the periods of such high water are short, the damage to the navigable rivers, if any should result, will be limited, and will in any case be less than the river can repair with its own scouring action.

The work of excavating the cut at Daguerre Point has been in progress for more than two years. About 650 000 cu. yd. of excavation have already been removed, and the work is now nearing completion.

A steam shovel having a 1½-cu. yd. dipper is used, the excavated material being removed, by two trains of about 10 cars each, to the dump at the north side of the entrance. An average of 30 000 cu. yd. per month is required under the contract, which will be completed by the close of the calendar year. The contract price of excavation is 23½ cents per cu. yd. of earth and 90 cents per cu. yd. of rock removed. The total cost of this cut will be somewhat more than $160 000.

Lip.—In order to prevent unusual scour through this cut, which must take nearly all the maximum flow of the Yuba River, amounting in floods to as much as 90 000 cu. ft. per sec., and perhaps more, it was necessary to construct an inlet wall or sill at the entrance. This is of masonry, and is sufficient to hold the bottom and banks in its vicinity during this extreme flow. Accordingly, contracts have been let for a reinforced concrete sill with a reinforced concrete abutment at the north end and with a rock face for an abutment at the south end. This work is now under construction. The length will be about 700 ft.

The estimated cost of the work is given in Table 3, the prices being those of the contracts now in force:

TABLE 3.

Excavation	2 200 cu. yd. earth.	at	$1.00	per yd.	$2 200
"	200 "	rock,	"	1.50	"	300
Back-fill..............	140 "		"	1.00	"	140
Embankment.........	300 "		"	0.75	"	225
Concrete.............	2 340 "		"	5.00	"	11 700
Plain steel bars.......	30 100 lb.		"	0.06	per lb.	1 806
Twisted steel bars	8 300 "		"	0.07	"	581
Excavation at entrance.	32 000 cu. yd.	earth,	"	0.45	per yd.	14 400
	1 000 "	rock,	"	0.45	"	450
Total						$31 802

FIG. 1.—TOP VIEW OF BARRIER; SECOND STEP NEARLY COMPLETED; RIVER PASSING AROUND FARTHER END.

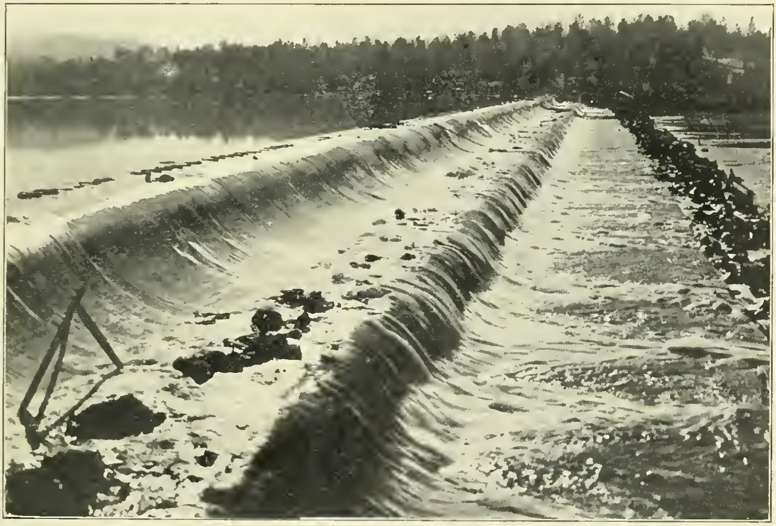


FIG. 2.—SECOND STEP OF BARRIER COMPLETED. DEBRIS ON STEPS WILL BE REMOVED BY FIRST HIGH WATER.

Cement is furnished by the Commission, and costs $2.25 per bbl. in Marysville and 41 cents per bbl. for hauling 11 miles to the site of the work.

The average cost, excluding abutments and the excavation of material in the entrance, will be approximately $35.40 per lin. ft. of length of the wall.

It was originally intended that the United States should build the high embankments extending across this part of the river, but before operations could be begun it was found that a gold dredging company had secured extensive mining rights in the vicinity, and that this promised to conflict decidedly with the plans of the Government. It was also learned that the company, in the exercise of its rights, proposed to dredge for gold in the river-bed just where the original settling basin had been located. After considerable controversy, an amicable agreement was reached by which the Commission was enabled to obtain a larger settling basin on the south side of the river in exchange for the location first adopted, certain deeds to property essential to the project were secured, and the dredging company also agreed to build the necessary embankments free of cost to the United States. The area mined was to be returned to the United States when it had been completely dredged, for such use as could be made of it. The embankments were of considerable value to the dredging company, as they protect its operations during high water, but they are built along lines proposed by the United States and save the cost of construction to the United States. The total length of embankment will be about 2½ miles. About ½ mile has been built already with only two dredges. Six more dredges are under construction, and will be placed on this embankment work as soon as completed. The embankments now being built are about 30 ft. high and from 200 to 300 ft. wide on the base. They will not be over-topped. These embankments will be completed, it is expected, by the close of 1906.

Training Walls.—Below Daguerre Point the debris in the river-bed is from 7 to 25 ft. deep, increasing from Marysville as one proceeds up stream, and forming a body about 2 miles wide, on an average, and more than 9 miles long. The project contemplates confining the river to a selected channel and leaving the remainder of the old bed undisturbed. Portions of the old bed may be used for

settling basins, but the unrestricted movement of the river over this abnormal width of bed is to be prevented. This will be effected by dikes at proper distances apart, which will contain the maximum flow. This will undoubtedly result in some scour in the selected channel until the equilibrium of the forces at work has been more nearly reached. This action, however, will all be in the direction of greater security from floods. Already, agreements have been made with two gold dredging companies to build 2 miles of these walls, lying on the south side of the channel below Daguerre Point, free of cost to the United States. The north wall for a corresponding length of 2 miles is now being built under contract by the Commission at a cost of 12.4 cents per cu. yd. of material in place. The total quantity in this north wall will be about 200 000 cu. yd. The distance between these training walls at the upper end is 2 000 ft. The distance apart will be adapted to the slope and the height of the walls in the lower reaches.

Other Rivers.—During the summer of 1905 a survey party was kept in the field surveying Bear River, with a view to its treatment along similar lines. The Bear has been surveyed from its mouth in the Feather up to the mouth of Greenhorn Creek, a distance of 32 miles, sites for dams have been examined, and a plan for work is now being formulated.

The American River will next be studied, and a plan for its treatment prepared.

When these other rivers have been remedied, the worst offenders will be rendered harmless, and the Commission can then turn its attention to the other problems of navigation and flood control in the Sacramento Valley.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

A NEW GRAVING DOCK AT NAGASAKI, JAPAN.

Discussion.*

By Charles Albertson, M. Am. Soc. C. E.

Charles Albertson, M. Am. Soc. C. E. (by letter).—The original graving dock in Japan was not a graving dock at all, in the present sense of the word. It was simply a sheltered cove or a sandy beach protected by nearby headlands from the severe winds and typhoons which occur at certain seasons. These beaches are found all along the coast lines of the four main, and the many lesser, islands which compose the Island Kingdom. Even to this day, the largest junks are beached in the old-time way, in order to clean the hulls and make minor repairs during the successive periods between high tides. Mr. Albertson.

For centuries the Japanese had been experienced in the control of water for purposes of irrigation, therefore it is quite probable that the next step in their docking operations was to build a small dam and gate, sufficient at least to protect the little inlets from the peak of the high tide.

From such small beginnings to the great graving dock described by Dr. Shiraishi is a tremendous leap, but the Japanese have shown themselves capable of doing just such things.

One would scarcely dare maintain that the necessity for such docks or the ability to design and build them could have come about except for contact with the nations of Europe and America. Their docks are based on the best foreign practice, but are slightly modified

* Continued from January, 1906, *Proceedings*. See October, 1905, *Proceedings* for paper on this subject by Naoji Shiraishi, M. Am. Soc. C. E.

Mr. Albertson. to suit the local demands and conditions. Nevertheless, it is no small matter to build a good dock in Japan, owing to the inexperienced, careless, heedless labor which must be used on such works. The laborer does his work in a very mechanical manner, and all his thinking must be done for him. It is a pleasure to know that Dr. Shiraishi has been entirely successful in the construction of his dock.

Japan is fortunate in having an indented, rather rocky, sloping shore line, which is naturally adapted to dock construction. The Japanese have made good use of these conditions, and most of their dozen or more large private docks are built directly on a rock foundation. They are all faced with native granite. The pump-houses are largely below ground, and contain electrically-driven, direct-connected, centrifugal pumping outfits. Dr. Shiraishi's article describes Japan's present standard practice.

Mr. Jacobs, in discussing the paper, stated that another gate in the middle of the dock would have been an improvement on the design, as one or two small vessels could have then been docked separately. While this might be true, ordinarily, if other docks in the immediate vicinity were not available, and but few large vessels were to be docked, it does not hold in the case of the Nagasaki dock. The owners, The Mitsu Bishi Company, have in operation two other docks and one patent marine railway. The dimensions of these smaller docks are: length on keel-blocks, 510 and 350 ft.; bottom entrance width, 77 and 53 ft.; and depth of water on keel-blocks, 26½ and 24 ft., respectively. The marine railway can accommodate a vessel of 1 000 tons. These, with the other small shipyards in the neighborhood, fully cover the docking requirements of Nagasaki's beautiful harbor.

In regard to cement, the writer shares the opinion which is general among foreigners in Japan, that the best Japanese cement is not equal to the best imported article. Some evidence of this is given by the Japanese themselves, in that the new dry dock, of the Kawasaki Dock Yard Company, Limited, at Kobe, was built with imported cement. This dock cost more than the big Nagasaki dock, although it is not nearly as large, since it measures 426 ft. from caisson to head wall toe, bottom entrance width 52 ft., and depth of sill 24 ft. The great cost was due to the company's shipbuilding plant having been located on bad ground, without considering dry-dock requirements. This necessitated piling and a large quantity of concrete. The Mitsu Bishi Company profited by the experience of the Kawasaki Company, and have now in use in Kobe Harbor a steel floating dock of 7 000 tons capacity.

Before giving up his residence in Japan, the writer saw the early stages of the construction of the Nagasaki dock. He is of the belief that, under the conditions existing then and there, only a little

more machinery could have been used either to advance the work or decrease the cost. The thought is that perhaps an addition to the limited hauling equipment, or even a further installation of some very simple type of conveying apparatus might have been desirable. This, probably, would not have decreased the cost to the contractor, although it should have reduced by a little the time taken in construction; but, then, time is not considered as valuable in Japan as it is in America. Mr. Albertson.

The writer feels that the credit for the design as well as the construction of this the greatest dock in the Far East properly belongs to Dr. Shiraishi, although he modestly refrains from giving even a clue as to his continued connection with the project.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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PAPERS AND DISCUSSIONS.

THE POSITION OF THE CONSTRUCTING ENGINEER, AND HIS DUTIES IN RELATION TO INSPECTION AND THE ENFORCEMENT OF CONTRACTS.

Discussion.*

By Messrs. James Smith Haring, W. D. Lovell, Benjamin Thompson, S. Bent Russell, Willard Beahan, W. A. Aiken, Augustus Smith and G. S. Bixby.

Mr. Haring. James Smith Haring, M. Am. Soc. C. E. (by letter).—The author seems to lay particular stress upon the "tact" which an inspector on construction should possess. The writer agrees with him, but would emphasize equally the fact that an inspector must be, also, thoroughly honest and thoroughly loyal to his chief or his principal. The writer has been a sufferer in cases where an exceptional amount of "tact" in his inspectors and an absence of honesty and loyalty have not only suffered the work to be compromised, but his own position to be jeopardized—the "tact" working for the interest of the contractors.

The personal equation of contractors is as material to courteous relations on work as the personality of the inspector. Some contractors are honest in their intentions and faithful in the perform-

* This discussion (of the paper by Albert J. Himes, M. Am. Soc. C. E., printed in *Proceedings* for November, 1905), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Communications on this subject received prior to March 30th, 1906, will be published subsequently.

ance of their obligations, no dereliction of duty or performance is a part of their creed, and yet they may have in their employ men actually performing the work who think they can perform or omit the performance of certain obligations in a manner that will save money for their employer, thereby advancing their own prospects. Necessarily, a careful inspector objects to such actions, and, unless the contractor is himself honest enough to see the justness of criticism, friction must arise, and the result soon determines the relations between the engineer and the contractor. Mr. Haring.

Chief engineers, themselves, are not always reasonable in their relations to either their inspectors or the contractor. The writer had occasion to observe this not long ago. Through a series of misfortunes, a contractor was doing a piece of work at a considerable loss, financially, but, notwithstanding this fact, when any reasonable request was made by the inspector for any particular item to be executed, it was done with alacrity and cheerfulness, the whole aim of the contractor and all his men being apparent in an attempt on their part to do the best work possible regardless of the cost. With all this evidence of good intention before him, the chief engineer, in several instances, cast suspicion on the work, both of the contractor and the inspector, in a manner so aggravating (and affecting matters so trifling) as to arouse the antagonism of the inspector, who was both honest and loyal, and the contractor, simply because it was unjust; yet the work, when completed, received the most positive commendation as being first-class in every particular, and it certainly deserved such praise.

On the other hand, the writer has had certain adverse experiences with contractors. In one particular instance the contractor started out with the avowed intention of laying all manner of pitfalls for the engineer and his inspectors, intending to catch them sufficiently at fault to find cause for making the contract void and laying the blame on the principals, behind the engineer, who acted upon his advice. In two other cases the incompetency (inexperience or financial inability) of the contractors was shown at the very beginning of the work, and it required not only "tact," firmness, and a knowledge of contract law, but a fight against political influence in the camp of his principals to enable the engineer to guide, not only the details of the work, but the persons who were paying for the work, out of grievous and sore tribulations, and to save his own reputation.

Too much cannot be said regarding the knowledge of the law of contracts to be possessed by the engineer. He must be able to protect his own rights, and unless acting for some corporation which maintains a legal department or retains regular counsel, he should be able to advise and direct those for whom he acts. In any condi-

Mr. Haring. tion, the engineer is charged with large responsibility; he is the active agency to take the responsibility for the work of his inspectors and assume it as his own; he is the target of the contractor and all his employees, and must stand their criticism and too often their abuse; he must frequently meet a combination of the contractor and his own employer or principal trying to place him in the wrong; so that his justification and defense must stand on indisputable grounds of right, or he is likely to fail utterly.

The condition which makes the engineer sick at heart, however, is to find honest effort on his part met by censure and condemnation in the light of the most positive evidence of the fact that his course is correct. The writer recalls a condition where he, as engineer, called to his aid inspectors whose capabilities, honesty and loyalty had been proven on other work. These inspectors did their duty against adverse circumstances. The work was for a municipality, and he regrets to say that he has spent whole sessions with the governing officials, who took up the time, badly needed for important business, in listening to the complaints of inexperienced contractors against these inspectors, simply because they had performed their duty conscientiously. The writer upheld them, and refused to countenance censure or removal. Finally, he himself was replaced by a more pliable man, who permitted the contractors to obtain payments for extra work to which they were not entitled, and who permitted the use of materials the writer had rejected.

In another instance the writer was engineer on a contract abandoned by the contractor because the engineer ordered an additional quantity of work done at a specific price named in the contract. The work was finished by the municipality under the terms of the contract, the writer acting as engineer, and a large deficiency for cost of construction has just been obtained by a jury trial in a suit to recover from the contractor.

Under any conditions, the engineer and his inspectors occupy responsible positions. If due consideration is not given in the selection of the inspectors, and there is not sufficient latitude, or absolute control of appointment, it too frequently occurs that competency is not the determining factor in the choice of men. A contractor is very soon able to "size up" the capabilities of the man who is to watch his work, and he acts accordingly.

As this is the age wherein women seem to be displacing men, the writer has often wondered whether the time will come when, to some degree, women will displace men in the engineering profession. In certain cases mentioned herein the proverbial "tact" of women might be found to be advantageous.

Mr. Lovell. W. D. Lovell, M. Am. Soc. C. E. (by letter).—This topic is one of general interest to both engineers and contractors. Undoubtedly,

"there is too much laxity in the preparation of specifications." It Mr. Lovell. has become the custom with many engineers to write their specifications hurriedly, or to have them prepared by an incompetent assistant, and then, to make sure that they have covered everything, to put in a general clause, as pointed out by the author. This blanket clause is an admission of the weakness of the specification, and its use should be discouraged. In making up his bid, the contractor depends upon the written specification to describe the work he is expected to perform and not upon the "general conditions" which may mean nothing, or a great deal, depending on the inspector.

In one instance, coming to the writer's notice, the specification relating to concrete seemed to be voluminous. Omitting the specification on cement, one thousand words were used in describing concrete, yet, when construction began, the inspector found it necessary, in order to get such work as he thought desirable, to hide behind a general clause, applicable to all divisions of the work, which stated that "all work must be done in a workmanlike manner." Relying on this blanket clause forced the contractor to increase the actual cost of the work 50%, and no extras were allowed. It is injustice like this which causes unpleasant relations between inspector and contractor, and brings the engineering profession into disrepute.

It often happens, especially on Government work, that the man who writes the specification has no part in the supervision of the work. The opinion of what is a "workmanlike manner," on the part of the man who wrote the specification, may differ widely from the ideas of the constructing engineer. The man who wrote may have had in mind a good class of commercial work with nothing fancy or finished about it. The inspector may happen to have decided opinions regarding the finishing of work, and may attempt to compel the contractor to do the work according to his ideas. He cannot do so by the specification on that particular subject, therefore it is necessary for him to call up the "general conditions," usually written in specifications, that "work must be done in a workmanlike manner" or "to the satisfaction of the engineer." This is not a fair interpretation of the specification or the contract, and yet, as is suggested by the author, a contractor very often submits to just such injustice because he cannot afford the time or the money to take the matter into court. Each specification, covering any one part of the work, should be written so that it should not be necessary for the inspector to rely on general clauses to support his requirements in regard to the character of the work, and "blanket clauses," which may be used as a gross injustice to the contractor, should be omitted, except to cover unforeseen contingencies.

Mr. Himes has mentioned the necessity of the inspector knowing his business, and being familiar with the conditions surrounding the

Mr. Lovell. contract. The successful inspector must have a practical knowledge of first-class work. The contractor often repeats the old saying, "I have no trouble with the inspector who knows his business." It is the man who is afraid of his own position—who does not really know whether he is right or wrong—who, by his unreasonable requirements on some points and his laxity on others, shows his lack of knowledge, loses the respect of his chief and the contractor, and proves a failure as a constructing engineer.

The following is a case in point: The work consisted in laying cast-iron water pipe. The inspector was diligent in chipping and tapping the castings, although he had a certified test of the material used in the pipe and a sworn statement that each length had been tested under a pressure of 300 lb. He was careful to see that the letters, indicating the manufacturer and the year in which the pipe was cast, were easily legible, and that the dust was wiped out of the pipe before it was laid; yet, when it came to the really important work of seeing that the pipe had a good uniform bearing in the trench, that the jute was driven back, and the lead joint deep, properly run, and thoroughly caulked, he paid no attention whatever.

A part of the line was through a corn field, where any settlement of the back-filling would make no difference, as the field would be plowed over in the following spring. The back-filling was to be done with a scraper, the earth wet down with water furnished by the city, and pumped at considerable expense from a deep well. The inspection of this back-filling was the most severe that could be imagined. The additional cost to the municipality, on account of pumping water alone, was considerably more than the salary of a qualified inspector.

Mr. Thompson. Benjamin Thompson, M. Am. Soc. C. E. (by letter).—Mr. Himes' paper treats of a very important subject, and the writer, without desiring to be critical, wishes to state that it is not so much fluency of diction that is needed in writing specifications as plain, definite, specific, concise, comprehensive statements. It sometimes happens that specifications are what might be called a literary effort, which is apt to confuse and bemuddle the contractor, who, more often than not, has had no more education than what is embraced in the "three R's." What he needs is plain writing, showing clearly and specifically what he is to do, when he is to do it, and what and how he is to be paid for it. If the engineer has not investigated the situation and the conditions surrounding the proposed work in detail, he cannot prepare fully what the contractor ought to have for his guidance. The more care in preliminaries the less difficulty with contractors, and the less embarrassment in explaining why the total cost exceeds the estimates, the latter being the standing indictment against civil engineers.

Mr. Thompson. The author's epigrammatic statement that "confinement on suspicion is fully as inconvenient as confinement on conviction," "should be indelibly engraved in the memory of every young engineer," as Mr. Wellington would have said. And the writer would like to add Whittier's "Ah, what a thin partition shuts out from the eyes of the curious world the knowledge of evil deeds done in darkness."

The author's question, "Has any one heard of a multi-millionaire engineer?" reminds the writer of what was said to him some time ago by a gentleman who had just paid $40 000 commission to a real estate agent for the sale of some property. "That man worked two or three months, and an engineer might work as many years or more to get the same amount."

The writer asked, "Why should the real estate agent receive so much higher pay for his services than the engineer for an equal or much larger amount of work to make the sale possible?" His reply was, "I don't know, unless it was because he got the money."

Inspectors or resident engineers on extensive work usually come from different sections of the country, and have different ideas as to good construction, and how to proceed in special difficulties. It seems to the writer that it would be a good plan for the chief engineer to have the engineers and inspectors come together at the beginning of the work and at various times during its progress for the examination and discussion of the specifications and their application to the special difficulties or problems which may be met. If this were done, the character of the work under their charge would be more uniformly good, contractors would feel that they were all treated about alike, the individual inspector or resident engineer would feel that he could take a position in which he would be sustained by the judgment of his comrades and his chief, with the least annoyance and trouble to the latter, and the chief engineer would be doing most valuable service, not only to his employers, but to those under him.

Mr. Russell. S. Bent Russell, M. Am. Soc. C. E. (by letter).—This subject covers much of the whole field of engineering, and so much may be said upon it that it is not easy to treat it well within the limits of a brief essay.

The broad question of the proper interpretation of engineering specifications is of such importance to engineers that it might well be classed with those subjects that should be brought up annually for discussion.

Consistency is of great importance, both in writing the specifications and in executing them. Not only should an engineer be consistent in his own practice, but neighboring engineers should advise with each other, and try to make their practice consistent.

Mr. Russell. Going a step further, there should be a certain amount of consistency aimed at among the members of the whole profession.

The dictum of the engineer should gain strength, and, indeed, often does gain strength, by the fact that it not only represents the judgment of a man especially trained for the work, but also in a measure represents the consensus of opinion of the whole engineering profession.

The standing of the profession will certainly be elevated by greater unity among engineers in the matter of engineering contracts. A great deal of missionary work is needed among engineers. They should be better informed as to what is the best practice, both in writing and in interpreting specifications.

The author is to be congratulated upon his successful opening of so important a discussion, and upon the many good points that he has brought up in the paper.

As to preparing specifications, engineering has now become such an advanced science, and the volume of engineering knowledge has become so vast, compared with the capacity of any individual, that, with little violence, it might be said that when an engineer starts to write a specification, he is working on a subject of which he has little or no personal knowledge. For example, he may have had much experience in building structures of steel and of masonry, and now be called upon in the line of the structure of timber. He is dependent upon the work of other engineers. His information must be gathered from books, periodicals, etc.

Perhaps the first question for the engineer to decide is how much time can be given to research work. It depends, of course, upon the importance of the case. Most errors in engineering specifications come from lack of time and information in preparing them.

In the writer's judgment, engineers more often give too little time than too much to specification writing. Many a lawsuit could have been avoided by a few minutes more time in the beginning. This is a point that could be thoroughly discussed by engineers with great advantage, and the position of the author is well taken as to better specifications being needed.

Perhaps the next question, with the engineer who starts to draw up specifications, is the state of the market. If contractors are known to be hungry for work, specifications, of course, may be severe. They may be written so that the contractor will be lucky if he gets his capital back.

On the other hand, if contractors are well supplied with work, they are usually very independent. The engineer must see that the specifications are more than fair to the contractor. Otherwise, he will get no bids, or excessive prices.

Coming now to the inspection of engineering work, as in the

drafting of specifications, it is found that the greatest difficulties come from ignorance. Young inspectors, of course, do not know what is customary practice in the line of work which they are called upon to judge. In their inspection, they are often at a great disadvantage, in that they know less about what is right and customary than the contractor himself, and yet they must expound the law and control his actions. Mr. Russell.

On the other hand, inspectors of more experience are often worse, on the whole, because they have been badly trained. An inspector who has served under a careless or incompetent engineer may be very unsatisfactory, owing to faults in his training. An old bricklayer does not always make the best inspector of brickwork. He is apt to be prejudiced by the kind of work he has been doing in his trade.

The number of inspectors or the quantity of work to be covered by one inspector is a matter of great import. One does not like to have the cost of inspection too high in proportion.

Engineers, as a class, should insist that the success of the work should never be imperiled by fear of criticism as to the cost of inspection.

At the outset the inspecting engineer may well bear in mind the state of the market when the contract was let, as mentioned previously, and thus infer the probable width of margin on which the contractor has to work. The keener the competition the sharper should be the inspection, as a rule. More often than not, the amount expended on inspection is below the point of greatest economic results.

Specifications which give arbitrary power to the engineer are too apt to put him in an improper state of mind. With the best intentions in the world, an engineer unconsciously becomes careless of the rights of the contractor and sub-contractors until he is confronted with a lawsuit. The engineer and inspector should guard carefully against this failing.

Perhaps the most important point mentioned by the author is "the practice of helping out the contractor." In the writer's judgment, the views of engineers on this point should be fully brought out, and a great effort be made to get the consensus of opinion of experienced engineers on the question. There is too much variation in the practice of engineers in this direction. Personally, the writer thinks that, in any work of a public nature, the engineer should never permit "helping out" the contractor. Obviously, it does no good to contractors as a class, and better results will be obtained with the practice ruled out altogether.

In private contracts, this practice should never be allowed, without full knowledge of all the parties interested.

Mr. Russell. It should be noted here that there is often a third party, not named in the contract, but interested in the result, financially or otherwise, and this party is depending upon the fairness and integrity of the engineer to protect his rights and see that the work is executed as planned and brought to a successful conclusion.

The engineer should keep the interests of the third party in mind, and guard them where it is proper he should, or advise such party of any proposed changes in the work.

A matter of similar bearing is the proper treatment of subcontractors. The engineer should be fully informed as to the status of such parties, and should know whether they are being fairly treated by the general contractor; and, where their interests are at stake, should keep them in mind, carefully avoiding just cause for complaint from them.

An error that the engineer should guard against especially is that of allowing the contractor to deceive himself by practicing a sort of confidence game upon him. The contractor is allowed to believe that he is to be "helped out" by the engineer at some later time, and thus he is kept in good humor and tractable, without the engineer really committing himself. This is sometimes the engineer's method of managing the contractor, but it is decidedly dangerous, and should not be followed.

Engineers should settle disputes as soon as possible, and should not defer them. There is room for a great deal of diplomacy in the management of contractors who are engaged upon different parts of the same work, so that each shall be made to think that he is being treated at least as well as any other contractor. To do this, under all the complications that come up in engineering work, is sometimes most difficult.

At times, where a contractor shows a disposition to make trouble, the engineer will make concessions to him that are of doubtful propriety, in order that, should a lawsuit come up later, he may show that the contractor has been treated liberally. This, of course, is treading on dangerous ground, and the engineer should be very careful not to go too far in this direction, as his motives may easily be misconstrued.

The question of covering up errors made by the engineer in writing specifications, or in the plans or instructions, is one that could be discussed with interest. How far may the engineer go with propriety in this direction? This question often arises in actual work, and is frequently of great import. It is doubtful, however, if any generalizations can be made for such cases that would not apply to the conduct of human life.

The writer hopes that the points thus briefly called to mind will add something to the interest of the discussion.

WILLARD BEAHAN, M. AM. SOC. C. E. (by letter).—This paper is an unusual one in both its field and scope, and will be useful to the profession. Engineering has been broadening much of late, and annexing new territory to the old province of mathematics and science. This paper is of the new learning upon which engineers are entering. It trenches much on law, and it has to do with human nature. In each direction an engineer always needs instruction, and to-day more than ever. Mr. Beahan.

The author states that "A contract is a meeting of minds," and that there must be a "clear and definite understanding of what is to be done." Now, this understanding of the two minds is the contract. The written instrument is but a reflection of it, and is sometimes clouded by legal phrase or ignorance of the niceties of technical terms. That printed blank which we fill out is not the contract, in the eyes of the law. This fact makes blanket clauses a confession that the minds have not clearly met, and hence the contract is faulty, and the door of misunderstanding and of litigation is confusedly thrown open.

Engineers differ as to whether a detailed contract and specifications which endeavors to cover all points fully and minutely is best; or whether a brief and general specification is best. The writer supposes that each engineer will ever favor that one of these two which is better in accord with his own temperament. Governments and municipalities favor the first kind, while corporations and firms very often favor the latter. Most engineers will agree that, in general, during the execution of work under contract, questions will arise which are at least on the border line of the terms of the contract, and will raise questions for adjustment. The engineer in charge is the one to whom these questions present themselves, and they are a crucial trial to him. With reference to these recurring questions the writer's instructions at the outset of his career, as given to him by the late D. W. Washburn, Chief Engineer of Construction on Mr. Gould's South-West System, were as follows: "You are, of course, employed and paid by the Company, but it is your duty to stand between the Company and the contractor, and say what is right in equity." Thirty years have almost passed, but to the writer the instructions still seem to be right. Common sense is good railroading, and honesty is the best policy, as well as the best politics. No contract can be drawn that makes the exercise of judgment unnecessary.

As engineers, we cannot compel a contractor to do work not really shown on the plans or in the specifications or in the contract. Some contractors have done harm to engineers by willingly doing such work just to cover some sins of omission. The whole is vicious. If a mistake is made and some work is left out, say so;

Mr. Beahan. it is better for all, and cheaper for the company, to handle this as extra work outside the contract. The engineer or inspector who accepts favors from the contractor is lost. Blanket clauses are put in contracts to cover just such careless work, but anyone at all familiar with the law of contracts knows very well that such attempts are futile, and this fact is clearly brought out in the paper. It is to be hoped that the quoted clause in the elevator and power-plant contract is the very limit of unwisdom in this direction.

"Absolutism has no place in business." One would think this were not true if he read some contracts, where it would seem that the engineer can do anything he pleases and pass any judgment he sees fit. But the Courts sometimes make short shrift of such contracts, and it is readily seen that a "meeting of minds" has not yet convened. In fact, a "printed form," when used in a contract, biases a jury and brands that contract as arbitrary. As a profession, we need to know how Courts and juries view us and our acts. We need to get out of our offices and drafting rooms and just "come down to earth" and mix with the multitude. By doing so we can better earn our salaries in our mature years.

It is indeed hard to say what is 89% and what is 91% when 90% is the mark at which inspections are passed. As inspectors, in our younger years will we not mark it 89% and in our later life mark it 91 per cent.? Can we ever detect that 2% of excellence? Clearly, we must use our honest experience and judgment. But to be called back and shown that some of our 89% product is as good as some of our 91% product is the act of a young or ill-tempered contractor. An old Irishman of some experience and sound heart was once inspecting cross-ties and was shown some rejected ties on one side of the pile that were a little better than some accepted ties on the other side of that pile. After a moment's study he went back and rejected them all. Did he do right?

The absent inspection of masonry, like the absent treatment of disease, is believed in by some, but they are in the minority. There is too much inspection which does not inspect, and it brings inspection into disrepute. To have one inspector over two gangs of masons five miles apart is as foolish as to have one foreman over two gangs. This evil arises from letting too many small contracts when the cost of inspection thereupon adds much to the unit cost. A better plan is to do these small jobs by a company force.

The case of the mill inspection of rails cited in the paper raises this question, naturally: May it not cost less to try to make first-rate rails than it will cost to make poorer ones and take the chances of their rejection? Some may have seen grading contractors haul logs and brush into an embankment when it would have cost less to burn the logs and brush and haul in earth instead. Cheating comes

to be a chronic disease at the last. The writer once knew a contractor to tell a falsehood wilfully when it was against his interests. His brother, who was the other member of the firm, said, "W. is a fool! W. will lie at sixty cents on the dollar when the truth would be worth par." We can form the habit of thinking that the truth is ever against us. Mr. Beahan.

An inspector, not sustained or "backed up" by his superior, is a man whose salary is money wasted through no fault of his. Honesty, capacity, and courage are essentials in inspectors, or in engineers in immediate charge of the work. Said a prominent chief engineer of a granger road to the writer a few years ago, "I have had two hundred and forty civil engineers of various grades on our work this summer and in not a single case has there been the least suspicion of dishonesty." In a quarter of a century of railroad work over much of the United States the writer has never seen a dishonest act by an engineer. He has had only one dishonest engineering employee—a paymaster, and under mitigating circumstances in a foreign country. Will our friends, the contractors, who are given to careless expression, please commit to memory these statements of the writer on this point? Capacity, however, is a rarer quality. The writer thinks that too much is expected of young engineers as inspectors. It is not fair to them. The writer fully agrees with the paper, but must say that, for an inspector of execution, rather than of manufactures, he prefers an experienced craftsman. For example, his best results in pile-driving have been in using an experienced pile-driver man as an inspector, rather than a young engineer. So, too, for a modern building, he prefers a first-class carpenter who can read a plan, rather than a young architect or engineer. For such work he prefers an old foreman, of unusual intelligence, who, perhaps, may be in poor health or crippled, but who possesses all his faculties. If he has been for a long time with the company, so much the better.

Courage is a quality of character, rather than of education or experience. Years ago a collegian was not thought to be courageous, but college athletics have changed all that. Moral rather than physical courage is the kind most required, but they go best together in this case. Bluster is the outward expression of pure cowardice. The brave are quiet and use few words. Of these three great qualities it may be said that, as a rule, honesty is a matter of course with a technical graduate; capacity can be trained into him, while courage he must have inherited, in the main.

The author has properly pointed out the fact that tact is an essential to greatness in the engineering profession. Tact is not taught in technical schools nor could it be. But a little tact might have been talked to us now and then to good advantage. We were

Mr. Beahan. taught intolerance, and were sometimes taught self-sufficiency. This is not seen in the later generations of students, however. In later years engineers must learn that tact which they ignored at first. Tact is ready money where talent is capital. As a profession we are at fault in not cultivating tactfulness. It is the new learning of our calling and the younger engineers should take up the study of it right away. Knowing a thing is not enough. One must also be able to make others and capital know it, too, and through one's own self as the instrumentality. Learn a thing, learn to tell it or write it, learn to do this convincingly, and, finally, learn to do so in a pleasing way.

Mr. Aiken. W. A. Aiken, M. Am. Soc. C. E.—It is very refreshing to find in this paper a clear note of appreciation of the real value of inspection; not only of the actual features of any piece of engineering construction, but particularly of the materials entering therein. It is unfortunately the fact that many engineers do not fully realize the true value of this latter and certainly equally important inspection, or, if realizing it in a general way, have neither the time nor the opportunity to acquaint themselves sufficiently with its infinite detail, and so unconsciously confound the mere matter of testing with real inspection, of which testing is only a very small part.

In the speaker's opinion, there is nothing more certain, when once thoroughly grasped by contact with a properly organized system of inspection, than that no engineer would ever think of using materials in construction unless previously they had been thoroughly inspected; for, of all the specialties of engineering technical work, no other has been so generally brought into disrepute, with those "who understand," by this age's spirit of commercialism, the dominance of which has set up everywhere the false standard of dollars and cents. Thus the cost of so-called inspection, rather than the quality of service, is unfortunately very often the conclusive argument in deciding a matter which in no way should be thus influenced. Inspection that does not inspect is absolutely worthless in itself, and is a complete waste of money. The mere matter of testing (relieving the manufacturer of responsibility, as it does in a great measure) is in many cases an absolute farce, in so far as determining the true worth of the material furnished, particularly because, as Mr. Himes states, it has unfortunately grown into a generally recognized practice that the tests submitted are to be of the manufacturer's selection.

The tonnage—in these days often involving a poorer quality of manufactured material—is another feature influencing largely the quality of inspection, unless this is carried out purely upon the basis of quality of service, instead of as a commercial enterprise. When

the manufacturing plants are crowded with orders, each being pressed for prompt delivery, and, to meet these demands, the manufacturers are putting all their efforts toward increasing their output, with the temptation to disregard its quality, an entirely different condition confronts the inspecting engineer than when orders are few and competition keen. The purchaser demanding shipment, the manufacturer, knowing that among his customers there are some whose own or commercially employed inspection is largely perfunctory, though these very customers (due to their ignorance of what inspection should consist) may not realize this, and knowing also that others are perfectly willing to accept the manufacturer's guaranty, are features of the conditions which are difficult to meet except by the strictest insistence on the specifications in all essential matters, and the exercise of clear judgment in the matter of concessions, if it is desired to obtain material complying even with admittedly fair requirements. It is under such conditions that what Mr. Himes designates as "the kicker" develops, but the speaker is satisfied that this is desirable, although, in his opinion, it is not generally necessary, and never in the case of a competent inspector, except in plants in which the methods are not "straight." Certainly, by ignoring conditions at the start, involving, as this must, the quality of inspection guaranteed (and surely, thereby, the quality of material contracted for), the inspector becomes *particeps criminis*, making it more difficult afterward to protest effectually against aggravated and intolerable conditions, and making him largely responsible for the attitude of many manufacturers toward proper supervision by the inspector. Mr. Aiken.

Competent inspectors, even when supposedly hyper-critical, are generally so only from the manufacturer's erroneous standpoint. The manufacturers may maintain that they are entitled to only such information and facilities as are supposedly customary and called for by the average inspector. This is not tenable. On the contrary, any information in the manufacturer's possession relative to the material to be inspected belongs by right to the purchaser's representative, and may be properly called for without his deserving the name of "kicker," even though his requests may be beyond those made by the average inspector. There is no conceivable reason for a manufacturer to refuse such information and facilities, as at times is done, except the one plea that it is not customary—and this is no reason at all—or the other reason that such information and facilities would enable the inspector to keep better run of the material, and this is worse than no reason at all. A manufacturer who has nothing to conceal never objects to furnishing any information relative to material under inspection. It is only in the case of questionable practices—and, of course, these are

Mr. Aiken. not general by any means—that the "kicker" is prominently developed, and for excellent reasons. Also, occasionally, where personal feelings enter into the business relations, the manufacturer deliberately and intentionally, as it were, develops the "kicker" by continually putting obstacles in the way of his properly performing his duty, with the object of bringing him into disrepute with his superior who cannot be personally acquainted with the details of every transaction. A conservative inspector can be sorely harassed by a petty-minded representative of a manufacturer, and the patience of a Job and the tact of a Talleyrand are then necessary to steer a course where duty is fully done and no opening is given for lawful objection to method.

The specious pleas of the manufacturer to influence an inspector even properly are many, and in a measure must be recognized from the former's standpoint as possibly allowable, but these must not influence. For instance, the manufacturer may request the acceptance of a small lot of rejected material on account of its size, which from the manufacturer's standpoint determines its importance; or he may request the acceptance of a similar larger lot of material because its rejection would entail a loss to him. He may also advance the additional pleas of busy times, the uncertainty when replacement can be made, and the always prominent plea that the construction will be delayed. Such arguments, in the case of structural steel, lose their weight in some mill practices; very bad ones they are, and in time they must be controlled, when it is understood that the test pieces furnished often do not represent any considerable quantity of material actually then rolled, though they are presumably the melts from which an order is to be rolled later, if the tests pass the specification requirements.

In the case of final rejection, the loss is really nominal at the worst, the material being applied to some other order where the specifications are less rigorous or the inspection less carefully made. All of which emphasizes the fact that testing is not inspection, no matter how carefully and conscientiously the former may be done. Complete and thorough acquaintance with the process of manufacture in all stages, and the assurance that the material finally used is that tested, is the only criterion whereby to determine proper inspection, and to obtain this there is no information relative to the material that may not be properly asked for and required by the inspector.

Under all the stresses to which he is subjected, nothing is more appreciated by him, nothing is more necessary to him, than the endorsement of his superior officer; and nothing, even from the most selfish standpoint, pays the engineer so well in securing the best service from competent men as the proper support of his inspectors.

While concessions may be properly made at times, in the judgment of a competent inspector or under general rulings of his chief, the idea that a reason must be found for the acceptance of material, whenever it fails to meet certain specifications, is too often the attitude of the manufacturer. The inspector in charge, knowing as he should, the use to which any material is to be put, must be the judge. Any other viewpoint is subversive and not to be tolerated. Mr. Aiken.

It is to be hoped that the discussion of this most important class of work will cause engineers to recognize more clearly the value of proper inspection, which is undoubtedly of benefit to the manufacturer, who should have common cause with the inspector in producing first-class material.

Augustus Smith, M. Am. Soc. C. E.—This paper is considerably broader than its title would indicate. Besides the relation of the inspector or constructing engineer to his work, it opens up the whole question of that broad branch of engineering, so to speak, of how to get one's idea executed—how to get what you want done. The speaker has seen this problem from the viewpoint of the contractor—the virtuous contractor, he hopes it will be understood—and therefore will ignore the inspector altogether. Mr. Smith.

He will confine his few remarks to an idea that at first sight may appear to be irrelevant, but which has been suggested by two statements made in the paper. The first statement is on page 580* where the author says:

"The theory of the law is held by its devotees to be the discouragement of litigation, and, in this respect, because of the expense and the numerous difficulties and delays in getting a final decision, the legal profession has attained a degree of perfection which engineers may not hope to equal."

The law is so perfect in the direction pointed out by the author that, though every contract is based on ultimate recourse to the Courts, no one who has actually tried the process would think of trying again, even if he lived long enough. It is so perfect that those who practice it are making little or no effort to improve it. At the end of the last century, when, it will be remembered, a general *résumé* of progress in all lines and professions was rather a popular subject, the Law was the only profession that had no progress to report.

The second statement is found on page 589.* It reads:

"There is much said about the relations existing between the parties to a contract and the engineer, it being generally held in the profession that his attitude should be strictly impartial and that he should be no less alert to guard the interests of the contractor than those of his employer. Such a condition is a pleasing fiction, quite flattering to the engineer and agreeable to the contractor."

* *Proceedings*, Am. Soc. C. E., for November, 1905.

Mr. Smith. This is indeed a fiction. Aside from the biasing influences pointed out by the author, disputes generally arise from poor specifications, and frequently from lack of knowledge by the man who prepared the specifications who is then assumed to be impartial in interpreting them. What man, even among contractors, can be depended upon to be impartial under such circumstances?

In order to avoid the Scylla of the Court and the Charybdis of the Engineer, some contracts provide for arbitration in case of dispute. The speaker has had no personal experience with the working of the arbitration clause, but understands that in general it is unsatisfactory.

Now for a "remedy." If the American Society of Civil Engineers found it permissible and expedient to elect with the other officers a contract committee, having cognizance of such disputes as Mr. Himes refers to, much as the regatta committee of a yacht club settles all questions of fair sailing, a great advance in "arbitration" would be made.

The membership of the American Society of Civil Engineers includes many contractors and many engineers individually professional who are employed by contracting firms. A decision of the contract committee of the American Society of Civil Engineers would command more respect from this class of disputants than a decision on engineering subjects by the Court of Appeals.

Expert legal testimony might be necessary at times, but let it be the lawyer before the Bench of Engineers on purely engineering subjects, instead of the engineer before a Bench of Lawyers who are generally quite uninterested and frequently half asleep.

It would be necessary, of course, to provide a scale of fees for such a committee, properly payable by the contestants, and, if found desirable, these could be made high enough to "discourage" litigation.

Mr. Bixby. G. S. Bixby, Esq.—Mr. Himes has taken up for discussion one of the most difficult and perplexing problems in the engineering profession. To outsiders the difficult features of the engineer's work seem to lie in logarithms and angles, in those terribly long lines of figures with signs of all sorts between them and over and under them, and in the mathematical features of the work generally; but, of course, those things are mere play to the engineer, and the speaker suspects that his real troubles begin when he is held responsible for money values, and when he is made a buffer between conflicting business interests.

This paper is a very valuable contribution, from the standpoint of practical engineering. The speaker knows personally that Mr. Himes began turning these things over in his mind a good many years ago, for when the speaker first knew him he was an engineer

on the Erie Canal, and even then had the idea that an engineer was supposed to work for the interests of his employer. Doubtless he had some troubles of his own growing out of the practical application of that idea. At any rate, some other people had troubles on his account. At that time he used to spend his spare time studying law, and he must have studied to good advantage, for his points of law seem to be generally well taken. One of the most valuable suggestions made in this paper is as to keeping a diary on inspection work. As a lawyer, the speaker will say that he has never seen a witness stumped on the stand when he had on hand for reference a record of events made in chronological order. Mr. Bixby.

A prominent feature of the work of the constructing engineer is that there is a tendency in practice to exact from him what is well nigh an impossibility. Wherever engineering is made up largely of work which is soon concealed, or the evidences of which are soon destroyed, it would seem that neither the expense nor the time allowed for inspection is ordinarily enough to enable an engineer to give positively the certificates which are theoretically required from him. One cannot get away from the fact that the contractor's inerest is not that of the owner; nor that the most honest contractor when pinched on his margins will exercise a tendency to pinch on his work. One must also remember that a contractor owes a duty to himself, sometimes to his creditors, and that sometimes he is a trustee.

Another point on public work, and often on private work, is that the certificating engineer has little to do with the choice of his inspectors. In a sense they are supposed to be his agents, but often they are independent employees, and it seems to be hard to exact from an engineer a certificate in the form which, as the speaker understands it, is ordinarily required that a thing is so and so as of his own knowledge, when he cannot see everything, and is dependent on others.

As a matter of justice, an engineer responsible for results ought to have at least a partial voice in the choice of sub-engineers and inspectors.

This paper refers very intelligently to the subject of perfection in materials and workmanship. Of course, it is known that there is no such thing as perfection, and specifications ought to have incorporated in them the permitted variations more than they do. It would be simply following current practice, but, if that is so, why should it not be expressed? In public works the conditions are frequently quite inflexible. Sticks of timber and pieces of iron and steel must vary, and yet, under a standard fixed for Government work, every defective bolt may be a nail in your coffin if it comes to light.

Mr. Bixby For instance, where hundreds or thousands of units are combined in a structure it may be unjust to require every piece to be of a standard character, and yet on public works the contract, the specifications, and often a statute unite in fixing a standard, and when there are such plain, specific provisions it is hard to invoke the doctrine of reasonableness or current practice.

There is a general opinion that a professional engineer on contract work is a kind of judicial officer, that his decision is like that of a Court, and that such decision must be made impartially between the parties. This is referred to in the paper on page 589.* The speaker does not believe that this theory has any standing in law, but thinks it arises from the fact that, as a matter of law, when, under a contract, a question is submitted to an engineer or an architect for decision, his decision cannot be an arbitrary one, but must be reasonable. In other words, the contract is to do a certain thing in a workmanlike manner, and if it is so done the inspector is bound so to decide. So, although the speaker's experience has taught him that the engineer, in practice, is very apt to be treated as an umpire, in reality he is not an umpire.

Not only is he employed and paid by one side only, but in all ordinary cases he would be subject to discharge or transfer by one side and not by the other.

This opens a very broad field for thought. The engineering profession is increasing in importance daily. The great works are multiplying so fast that we lose track of them and a hundred million dollars is becoming an ordinary sum.

The public is becoming more and more inclined to undertake these vast enterprises, and, in doing so, it is more and more dependent on the engineer. The contractors' interests also require protection. On the face, the contractor would seem to be at a disadvantage, for, while the party of the first part can ordinarily call on the engineer for such protection as may be needed, the contractor, if he disputes the reasonableness of the engineer's decision, must invoke the aid of a Court.

It is the greatest possible compliment to the engineering profession that so comparatively few lawsuits arise on important works. Of course, they arise often enough, and when they do they illustrate the difficult questions in hand.

The speaker has in mind one lawsuit now pending between a railroad and the contractor, which grew out of the holding up of final payments which would have been acceptable to the contractor at about $100 000. In the suit claim is made for $1 500 000 or more, and the contractor expects to make good his claim to many times the sum he would have accepted on the completion of the work.

* *Proceedings*, Am. Soc. C. E., for November, 1905.

Here is a question which often comes up: Suppose on important work a question arises which is obscure as to its solution, but nevertheless vital in the progress of the work. That is—the scope of the question is defined, it has to be decided, and engineering opinion relating thereto is divergent. Perhaps it has been in the Courts and has been decided in different ways, or perhaps it has been left undecided. What is the engineer to do, and how is he to be protected? The parties can fight it out afterward, perhaps, but it places the engineer in a very difficult position. Mr. Bixby.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
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PAPERS AND DISCUSSIONS.

THE INSPECTION OF TREATMENT FOR THE PROTECTION OF TIMBER BY THE INJECTION OF CREOSOTE OIL.

Discussion.*

By John B. Lindsey, Jr., Assoc. M. Am. Soc. C. E.

Mr. Lindsey. John B. Lindsey, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Stanford emphasizes the point of view that, contracts for the treatment of timber and piling being based on the weight of oil injected per cubic foot, the logical basis for inspection should be the weight injected per cubic foot. It is extremely doubtful, however, whether any system short of actually weighing the entire cylinder load of timber, both untreated and treated, would be considered favorably either by the management of the treating plants or by those who use creosoted material. The structure of different pieces of timber varies too greatly to permit the adoption of any average sample-piece method. The writer has noticed a marked difference in the treatment of different sections of the same pile, and occasionally a marked difference in the penetration on the same section. In most cases the closer grain of the less penetrated section explained the difference, but in some cases there was no apparent cause to explain the lack of uniform treatment.

The teredo-eaten pile at Pensacola, described by Mr. Stanford, with a defective 90° sector of sap wood, seems to be a practical example of the uncertainty of securing uniform treatment along

* Continued from January, 1906, *Proceedings*. See November, 1905, *Proceedings* for paper on this subject by H. R. Stanford, M. Am. Soc. C. E.

the entire length of a pile where the percentage of sap wood to heart wood is much more constant at every cross-section than in the case of a sawed stick, and where conditions as to steam pressure, degree of vacuum and oil pressure were identical. Mr. Lindsey.

How much greater, then, does the uncertainty of uniform treatment become when the treatment of individual sticks is considered. The stick with the greater proportion of sap to heart wood, and more open grain, may receive a 20% greater injection than another stick in the same cylinder load.

Clause 4 of Mr. Stanford's proposed specifications, to insure the uniform size and structure of the pieces in each cylinder load, although entirely approved by the writer, is a difficult one to carry out fully in actual practice. This difficulty is especially great at present, as there is such an unprecedented demand for all classes of structural timber and lumber.

The percentage of the total weight to be assumed as becoming seasoned during the steaming and vacuum periods would be another vexing problem, difficult to determine equitably. The question as to whether 8% or 15% of the weight of green sticks in a load would be seasoned would mean a difference of about 4 lb. in the treatment. Such a consideration, in a plant not operated conscientiously, would have a tendency to reduce the effectiveness of the steaming and vacuum periods, affecting economy of fuel and securing a greater estimated injection of oil than actually made.

In the future the greater portion of creosoted material needed will consist of cross-ties, telegraph and telephone poles and cross-arms. Such standard-size stock will doubtless be, to a large extent, air-seasoned before treatment. Operating upon air-seasoned stock, the steaming period would be reduced to a brief interval for sterilizing the timber, or perhaps be entirely omitted. The seasoning percentage factor would then be reduced to a minimum, and with stock, say, not more than 50 ft. long it would be entirely practicable to weigh each section of the load before and after treatment. The injection could thus be determined by the estimate of the weight of oil injected in the entire cylinder load.

O. Chanute, Past-President, Am. Soc. C. E., in his valuable paper on "The Preservation of Railway Ties in Europe,"* gives the following interesting evidence of the care exercised by the German plants in treating ties:

"The most notable thing in Germany is the painstaking care with which every operation is performed. The ties are not treated until they are thoroughly seasoned, this generally takes six months to one year after cutting and piling in open piles in the yards, most of which yards will hold one year's supply. The chemicals are tested constantly, a laboratory being attached to each plant, each

* *Transactions*, Am. Soc. C. E., Vol. XLV, p. 498.

Mr. Lindsey. buggy load of 32 ties is weighed before and after treatment, to make sure that the ties have absorbed enough and every little while each individual tie of a buggy load is weighed in and out."

Present Methods.—The methods used to determine the quantity of creosote oil injected into timber and piling by most of the timber-treating plants in the United States are much the same, and, in all the plants of which the writer has personal knowledge, depend upon the quantity of oil taken from a supply tank, as determined by the position of a float in the tank. This float is connected to a sliding indicator on the scale board in such a way that the depth of oil in the tank is recorded in feet and tenths.

The readings of the gauge taken during the treatment of a load are usually as follows:

A-Reading = Depth of oil in the supply tank before any oil is admitted to the treating cylinder;

B-Reading = Depth of oil in the supply tank at the instant the cylinder is filled; the oil taken from the supply tank equals the content of the cylinder when empty, less the volume of the timber load;

C-Reading = Depth of oil in the supply tank after additional oil has been pumped into the cylinder with the pressure pump; the additional oil forced into the cylinder equals the quantity of oil which it is calculated the timber load is to receive;

D-Reading = Depth of oil in the supply tank when the surplus oil is returned to the supply tank from the cylinder.

The difference between Readings *B* and *C* is the estimated quantity of oil the load is to receive. Lack of accuracy in the gauge mechanism is very objectionable, and should be reduced to a minimum; however, this defect is as likely to increase as to decrease the injection.

The lack of refinement due to the use of a measuring tank of large horizontal capacity might be avoided by having a supply tank of small diameter, say, 6 ft., to measure more accurately the quantity of oil forced into the cylinder by the pressure pump after the cylinder had been filled from the large supply tank; however, with the use of two supply tanks, it becomes more difficult to secure a satisfactory check on the quantity of oil used than can be ascertained when only one tank is in service.

The losses due to leaking pipes, valves and cylinder heads are extremely small in a plant where a proper degree of attention is given to the equipment. The loss due to leaking valves is the only one not readily observable, and close inspection of the condition of the valves should be made at regular intervals.

The quantity of oil absorbed by timber during the time the cylinder is filling with oil may be considerable, with well-seasoned stock, especially with such materials as paving blocks. After the *C*-Reading is recorded, and the pressure on the oil cylinder is released, preparatory to emptying the surplus oil from the cylinder, it is uncertain whether the entire quantity of oil injected into the load remains in the timber. To form a check on these probable inaccuracies, the *D*-Reading should be subtracted from the *A*-Reading, to determine the actual quantity of oil used. The difference gives the actual impregnation the entire load has received, provided there is no loss from leaking valves, pipes, or cylinder heads. Mr. Lindsey.

Underground supply tanks or dumping tanks are objectionable unless they are in a cellar and permit the inspection of all tank connections.

Where a plant is equipped with an elevated supply tank, the cylinder is usually filled with oil through a 10 or 12-in. pipe connection. The pipe from the tank to the pressure pump is usually from 3 to 4 in. in diameter. By returning the oil, by compressed air, after treatment, to an elevated supply tank through a 10-in. connection, the supply tank, the treating cylinder, and the connecting pipe lines could be fully examined by the inspector during the course of the treatment. With such arrangements, an intelligent inspector, after careful study of the equipment of the plant, should be able to keep a reliable check on the conscientious and intelligent operation of the treatment. This supervision would generally require day and night inspectors.

Any changes in the present methods which will place the creosoting of timber on a more precise and scientific basis, and afford the fullest possible opportunity for intelligent inspection, will be welcomed by all who have at heart the proper interests of the business.

It appears to be entirely practicable to place the inspection of air-seasoned stock less than 50 ft. in length upon a weight basis; however, in the treatment of green lumber, piling of any length, and air-seasoned material more than 50 ft. in length, the tank system of measurement, by gauging the injection of oil, will probably remain in use. Effort should be made to abandon the use of underground dumping or supply tanks, and to simplify, as far as practicable, all oil-pipe connections between these tanks and the treating cylinders. The treatment which the material is to receive may be determined by the difference between the *A* and *D*-Readings of the gauge.

Quality of the Oil.—The presence of water in the oil should be carefully guarded against. None of the creosoting plants have stills or the oil manufacturer's proper equipment in order to free the oil from water entirely, and they have to depend largely upon

Mr. Lindsey. the settling method. This consists in heating the mixture to, say, 180° fahr., then chilling it and allowing the water to float to the surface, where it can be discharged through a connection in the side of the tank. Steam is kept circulating almost continuously through the heating coils in the treating cylinders and the supply tank. It is of the utmost importance that these coils be tight, and that any leaks which may occur be closed promptly. The heating coils in the storage tanks, through which it is necessary to circulate steam when oil is to be drawn from the tank, should be examined from time to time, in order to avoid unnoticed leakage.

Oil received in barrels should be dumped promptly to avoid leakage. If the barrels are stored in the yard for some time considerable rain water will seep through the heads of the barrels.

The requirements that no oil with more than 8% of water be used, and that any excess of water between 2½ and 8% be compensated for by a proportionately greater injection of oil into the load, are reasonable, and should insure good work.

Specifications as to the quality of the oil are at present based largely on a distillation process. There is some difference of opinion as to whether it is advisable to allow a small percentage which will boil below 210°, or to exclude this light oil entirely. There is much difference of opinion as to whether it is desirable to have the distillate between 210 and 235° cent., 20% or 40%, or an intermediate percentage. It is certainly desirable to determine, as nearly as practicable, the most effective quality of oil necessary to preserve timber. The specification will be modified generally, however, by commercial necessities. After the coal-tar manufacturer has extracted from his tar all the higher-priced products, the residue or creosote oil is sold to the timber-treating plants. If the purchaser of creosoted lumber is informed that specifying a maximum distillate to 235° cent., of 25%, instead of 45%, will mean a 20% increase in the price of his treated timber, he is very apt to change his specification, feeling doubtful whether the increase in price is compensated for by the better quality of oil. The consulting engineer of one of the largest purchasers of creosoted timber in the United States called for bids in December, 1904, to furnish the material needed by his client during 1905, and specified that the entire distillate, up to 235° cent., must not be greater than 30%, a quality of oil similar to that proposed by Mr. Stanford. Before the date of the letting of this contract the engineer advised all bidders that he had found it necessary to revise his specification in order to avoid excessive cost to his client. The revised specification allowed a maximum distillate of 60% up to 235° cent.

When the coal-tar manufacturer finds use for a portion of the present excess of oil boiling between 200 and 235° cent., it will be

practicable, without excessive extra cost, to conform to the specification proposed by Mr. Stanford. Mr. Lindsey.

In determining the steam pressure and the length of the steaming period to be used in the treatment, the quantity of oil to be injected, as well as the size of the material, should be considered. According to the investigation of Dr. Hermann von Schrenk, of the United States Department of Forestry, the strength of creosoted material is affected, not only by the heat used during treatment, but also to some extent by the oil injected. The experiments indicated that an injection of creosote oil weakened the stick to the same extent as the impregnation of an equal quantity of water.

Mr. Stanford's observation, that heart wood is practically impervious to oil, probably applies mainly to long-leaf yellow pine piling, and particularly to the butt half of the pile, where the sap ring receives practically the entire impregnation. The writer has seen a section, from 10 to 15 ft. from the top of a long-leaf pine pile, completely saturated with oil. Where lumber is treated the impregnation in long-leaf heart pieces is generally from ½ to 1½ in. Short-leaf open-grain pine is best adapted to receive a satisfactory impregnation, and pine of this class should be secured for treatment if practicable.

The writer has an 8 by 16 in. yellow pine stringer, creosoted by Mr. J. W. Putnam at the West Pascagoula Plant in 1877, which was completely impregnated with oil at a section 3 ft. from the end of the stick. This stringer stood service in the West Pascagoula Bridge for more than 27 years. Such impregnation, in long-leaf yellow pine heart material, is unusual, however. Mr. Putnam describes the treatment of this bridge material for the New Orleans and Mobile Railroad in his letter to the Committee of this Society which, on June 24th, 1885, made a report on the preservation of timber.

It would be interesting to know if the plant, at which Mr. Stanford inspected the treatment of 80-ft. piling where such deficient treatment was secured by the tank measurement method of injection, was equipped to ascertain the actual quantity of oil used in treatment by taking the difference between the *A* and *D*-Readings.

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PAPERS AND DISCUSSIONS.

THE CHANGES AT THE NEW CROTON DAM.

Discussion.*

By Messrs. William R. Hill and Frederic P. Stearns.

Mr. Hill. William R. Hill, M. Am. Soc. C. E.—The proper plan and specification for a reservoir embankment is a subject susceptible of many conflicting opinions. The formulation of such a plan depends entirely upon the condition existing at the site of each particular structure. In the main, these conditions are: the character of the natural foundation, and the character of the earth available to make the embankment. With this information at hand, the question of the necessity of a core-wall arises, and, if it is required, its character and dimensions must be determined. Then, as to the embankment itself, its height, width, slopes, paving and mode of construction must be fixed. To determine these important questions, the engineer must be guided entirely by his judgment, based upon experience and study of similar structures throughout the world. He might apply to science in vain, for he could get no help, as the efficiency of a reservoir embankment is not subject to computation; hence, it is not unreasonable to expect that conflicting opinions will arise as to the efficiency of a plan of a reservoir embankment.

Although this paper is entitled "Changes at the New Croton Dam," it treats of only one of the several changes that were made in

* This discussion (of the paper by Charles S. Gowen, M. Am. Soc. C. E., printed in *Proceedings* for December, 1905), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.
Communications on this subject received prior to March 30th, 1906, will be published subsequently.

the plan. To describe the structure briefly, it is composed of three distinct features, the spillway at the north end, the main stone dam, and the embankment with a core-wall at the south end. These, according to the original plan, had lengths of 1 000 ft., 600 ft. and 568 ft., respectively, making the total length of the structure 2 168 ft. Had the dam been built according to the original plan, the core-wall at the junction with the main stone dam would have had a height of 230 ft. Mr. Hill.

The first important change in the plan of this structure was made on September 16th, 1896, during the progress of the work. It consisted in extending the main stone dam a further distance of 110 ft., in substitution of the embankment and core-wall. That change was at once received with favor, and was carried out in the construction without any discussion whatsoever. It was of exactly the same nature and made for the same purpose as the change under consideration; that is, the main stone dam was extended in each case, with the sole object of reducing the height of the embankment and core-wall; and yet, while the first change materially increased the cost and delayed the completion of the work, it was not as effective as the change under consideration, as it resulted in reducing the height of the core-wall only 30 ft., still leaving it with the unprecedented height of 200 ft.

On January 1st, 1900, when the speaker assumed the responsibility for this work, the foundation of the stone dam had been completed to the surface of the ground, and the core-wall was completed, excepting the stretch under consideration, which lacked about 60 ft. of its height.

In the spring of 1901, the speaker's attention was called to five slight cracks in the core-wall, all within a distance of 100 ft. Relating to these, Mr. Gowen's paper states:

"It was so evident that they were due to changes of temperature and, possibly, to some extent, to shrinkage of the setting mortar, that they were not given serious consideration until Mr. Hill's attention was called to them, and by him they were considered so serious that his first report and recommendation that the core-wall be removed were very largely based upon them."

In reply to this, the speaker would state that he cannot concur in the opinion that the cracks were due to changes in temperature, as he could not expect contraction cracks to occur so closely together as five within a distance of 100 ft.; neither could he believe that they were caused by the shrinkage of the setting mortar, as such cracks could not extend through the wall, as they did in this case. But let the cause of the cracks be what it may, the cracks themselves were given importance by the speaker only inasmuch as they led him to a closer study of the plan, which study brought to light the really

Mr. Hill. objectionable features, as shown by his report to the Aqueduct Commissioners, dated May 15th, 1901, wherein he reported that the core-wall was cracked, pointed out the objectionable features, and recommended that they appoint a committee of engineers to pass upon the adequacy of the plan. The following is quoted from that report:

"Even though there were no cracks, I consider that it would be unwise to complete the structure under the present plan, as I consider it would be an experiment."

Hence, the recommendation to remove the core-wall was not based on the existence of the cracks, but solely upon the opinion that the plan was inadequate. The concluding paragraph of that report is as follows:

"I make this recommendation after carefully studying the situation and plan, and I know that I am absolutely right, but, still I feel that it is due to you, as well as to myself, that we should be fortified by the opinion of three prominent engineers in this most important matter, and I respectfully ask you to take the necessary action."

The Commissioners, after personal investigation, agreed to this and appointed a committee of expert engineers, consisting of Messrs. J. J. R. Croes, Past-President, Am. Soc. C. E.; Edwin F. Smith, M. Am. Soc. C. E., Chief Engineer of the Schuylkill Navigation Company; and Elnathan Sweet, M. Am. Soc. C. E., former Engineer of the State of New York. The committee, after making an investigation, reported unanimously recommending the removal of the core-wall and the extension of the stone dam.

The general public will no doubt feel that great weight has been added to these conclusions by the concurrence of the eminent engineers, William H. Burr, M. Am. Soc. C. E., occupying the Chair of Engineering of Columbia University, and Nelson P. Lewis, M. Am. Soc. C. E., Chief Engineer of the Board of Estimate and Apportionment of the City of New York, both of whom had been asked by Mayor Low to investigate and report thereon. On April 16th, 1902, the Aqueduct Commissioners resolved to remove the embankment and core-wall and to continue the main stone dam.

Mr. Gowen's paper contains two general contentions; one, that the plan of September 16th, 1896, was adequate; the other, that the natural foundation of the core-wall was safe.

To take up the first contention, that is, of the adequacy of the plan. This paramount question is treated in a brief manner, and without giving a clear description of the part of the plan under consideration. The only cross-section accompanying the paper is one of the embankment and core-wall at a point about 170 ft. from the end of the stone dam. In reference to this point, it states that

the original ground is only 20 ft. below ordinary high-water mark, and that the core-wall, 110 ft. high, was built in a trench 80 ft. deep. Mr. Hill.

The paper contains what he designates as a developed section, and this also passes through the core-wall at the same point, that is, about 170 ft. from the end of the stone dam, and, after passing through the core-wall, the section then follows on the so-called line of least resistance to the pressure and passage of water under or through the core-wall. This line is not a straight line at right angles to the structure, for it makes an abrupt angle on each side of the wall, both deflecting northerly; in fact, the up-stream line follows the top of the embankment to its end, and these two lines on opposite sides of the wall diverge from each other at an angle of only 45 degrees. On this crooked section, the author states:

"The thickness of the bank at ordinary high-water elevation is about 200 ft. and the thickness of the core-wall given at Elevation 80, if the hardpan be taken into account and included, is about 300 ft."

Here, it might be interesting to note that, on a true cross-section, the bank was to be 30 ft. thick at the top, while the wall at the elevation noted was to be only 17 ft. thick. The paper also states that, on the developed section, the slope of the embankment on the up-stream side is about 4 to 1. This is a mistake, as both the contract drawings and the plan accompanying the paper itself show the slope to be only 2 to 1.

Accompanying the paper is a plan showing in outline the section of embankment and core-wall between the end of the dam and the gate-house, a distance of about 275 ft., and a profile of the same showing the original ground and rock surface and the rock surface as excavated for the core-wall foundation.

Only two reasons are given to support the contention that the plan was adequate: One is a reference to a mass of hardpan covering the southern slope of the valley; the other is a denial of a statement that flowage of water is likely to occur along the face of the core-wall. What might be termed another reason is a citation of several dams that have been successfully built to heights ranging from 100 to 120 ft. As to the success of such structures, the paper contains the following:

"And it is a matter of common knowledge to those interested that there is no record extant of the failure of an earthen dam with a core-wall due to filtration through or under the wall and the consequent movement of the down-stream bank."

In reply to this the speaker would state that the records show that the Mill River Reservoir Dam, at Williamsburgh, Mass., burst on May 16th, 1874. It was an earthen dam, with a masonry core-

Mr. Hill. wall 600 ft. long and 43 ft. high. Water found its way under the core-wall and destroyed the embankment. The reservoir was suddenly emptied into a narrow valley, causing the loss of 140 lives and the destruction of about $1 000 000 worth of property.

The author also makes a reference to two rejoinders to the report of the Committee of Expert Engineers, when, without giving any information as to the contents of those rejoinders, his deduction from them is that they would seem to have covered completely and fully all the points advanced by the expert engineers and disposed of their conclusions effectively.

The foregoing constitutes all the information contained in the paper, concerning the dimensions and general features of the structure, and the reasons to support the contention that the plan was adequate. Upon these, so far as the paper is concerned, is based the conclusion:

"That the City of New York has expended unnecessarily nearly $1 000 000 and has failed to utilize at least two years of valuable time during which these changes at the New Croton Dam were being carried out."

The speaker, before presenting his view regarding the stability of the plan, deems it necessary to give the following brief description of the part of the embankment and core-wall under consideration. It extended from the end of the stone dam a distance of about 275 ft. to a gate-house built in the embankment. The core-wall at the end of the stone dam, as before stated, was to have a height of 200 ft., and, at the gate-house, a height of 90 ft. The embankment was to be 30 ft. wide at the top, with sides sloping in the ratio of 2 horizontal to 1 vertical. The lower portion of the inner slope, to a height of 16 ft. below the crest of the spillway, was to be paved with stone, 18 in. thick, laid dry, upon 12 in. of broken stone; and, on the upper part of the slope, to a height of 12 ft. above the crest of the spillway, the paving stone was to be 2 ft. thick, upon 18 in. of broken stone. The core-wall in the center of the embankment was to be 4 ft. higher than the crest of the spillway, 6 ft. wide at the top and increasing to 18 ft. at a depth of 136 ft., then it had the same width to the base. The high end of the core-wall had been built in a wide pit. That was a necessary excavation for the end of the stone dam, which was 164 ft. wide at the base, while the core-wall was only 18 ft. at its base. The slope of this pit extended southerly along the line of the core-wall for a distance of 150 ft.; thus the core-wall at its highest end was not built in a narrow trench below the surface of the ground, as is usual in ordinary cases. The outline of this great pit is shown on the plan accompanying the paper, and is marked "Top of Excavation."

There are in the plan three objectionable features which in-

fluenced the speaker to recommend the removal of the embankment and core-wall. They are as follows: First, the excessive height, narrow base, and unstable foundation of the embankment; second, the great height of the core-wall; and, third, the means afforded water to reach the core-wall. Mr. Hill.

To take up the first, the embankment: It was to be 150 ft. high, and only 650 ft. thick at the base. This section would be not only about 30% higher than any heretofore built, but, in comparison with other high embankments, its base was narrow for its height. As an example, the Amawalk Dam, which forms one of the upper Croton Reservoirs, while only about half the height, 85 ft., yet has a base wider than that of this embankment of unprecedented height; and, further, this embankment was hazardous because of the unstable nature of its foundation. It was founded over a great refilled pit, which was 360 ft. wide at the top, 170 ft. at the base and 70 ft. deep. This pit was a necessary excavation for the foundation of the end of the stone dam, which was 164 ft. wide at the base, as before stated. It would be impossible to refill this pit as compactly as the original hardpan; hence the safety of the reservoir was dependent not only on an embankment of a problematic section, but this problematic section rested upon an unstable foundation.

The second of the objections: The core-wall of this embankment was to have the great height of 200 ft. and with no lateral protection or support whatsoever from the original ground, as the artificially placed earth on each side of the wall in this wide pit had the height of the wall itself, 200 ft. The natural hardpan would afford no protection whatsoever here, inasmuch as it had been excavated to its entire depth; in fact, the underlying rock had been removed to a depth of about 15 ft. Considering the height of the wall, and this in artificially placed earth, it could be but an experimental structure, inasmuch as it would be about twice the height of any heretofore built.

The third objection, the means afforded the water to reach the core-wall: This is another serious objection, as the water, by starting at the end of the embankment in the reservoir and following between the face of the stone dam and the embankment, would inevitably reach the core-wall. It would be impossible to puddle or otherwise compact the embankment against the dam to prevent this, as settlement would surely follow in any embankment of this great height, and the settlement of the material under the projecting parts of the rock-faced masonry would leave cavities for the passage of water. This objectionable feature here exists because of the combination of a stone dam and an embankment, while it could not exist in either a continuous stone dam or, on the other hand, a continuous embankment and core-wall.

Mr. Hill. A fourth objection might here be stated, namely, the permeable and light character of the earth of which the embankment was made. Relating to this material, the Committee of Engineers reported:

"It is permeable to water under any head from 3 to 150 ft., and, when exposed to the direct action of water, it disintegrates and assumes a flat slope, the surface of which is best described as slimy."

Thus it will be seen that the safety of this reservoir was dependent, not only upon an embankment made of permeable material and of a problematic section resting upon an unstable foundation, but also upon a core-wall of phenomenal height, unprotected and unsupported by original soil and attended with the greatest of all possible risks; that is, the means afforded water to reach the center of the embankment against the core-wall. Such a structure, in the speaker's opinion, cannot be regarded as anything but an experiment, as it is abnormal and unprecedented in all its dangerous features. Thus, as the speaker was thoroughly convinced that the plan was inadequate, he was left no alternative but to condemn it.

Before closing, the speaker wishes to state that he has no desire to discuss the contention that the natural foundation of the core-wall was safe, as he wishes to maintain the stand he took at first; that is, that the plan of the structure itself was faulty, without considering the physical conditions existing below the base of the core-wall, and that the modification of the plan has resulted in the completion of the structure in keeping with the report of the Board of Expert Engineers, consisting of Messrs. J. J. R. Croes, Joseph P. Davis and William F. Shunk, who, in 1888, recommended that the Quaker Bridge Dam, for which this is a substitute, be a stone structure from end to end.

Mr. Stearns FREDERIC P. STEARNS, PRESIDENT, AM. SOC. C. E. (by letter).—Mr. Gowen's paper presents in a very clear way the conditions surrounding the dike of questionable limestone found at the southerly end of the dam, and he gives convincing reasons in support of the view that the construction, as originally planned and, to a large extent, executed at that place, was entirely safe. The writer has never examined this limestone, but he has had occasion to make tests of the bearing capacity of other soft rock, using for the purpose apparatus copied from that used for testing foundations at the New Croton Dam, and was surprised to find what a great difference there was between a very soft rock and the hardest and most compact earth. Therefore, he would expect any rock found in this section of the country to support a weight equal to that of a masonry core-wall.

The permeability of a rock foundation, where the rock is of compact texture, depends upon the presence of seams or other passages

for water, and not in any degree upon whether the rock is hard or soft. Mr. Stearns.

In a case like that described, where the core-wall was built in a narrow trench cut in firm hardpan extending to the rock, and where there was also provided an embankment of fine clayey material of great dimensions, there must be taken into account the resistance of this earth to seepage and to water pressure.

It is quite often the case that an embankment built of earth containing a sufficient proportion of fine particles is as nearly water-tight as a concrete or other masonry core-wall, but this is not recognized in all instances, possibly because the concrete has so much greater strength.

The Board of Expert Engineers, who recommended the changes at the New Croton Dam, caused many borings to be made in the embankments of the dams of the Croton system, and, in a majority of cases, the line of saturation determined by the investigations indicated no greater resistance to seepage or percolation at the core-wall than in the embankment of earth.

In view of the character of the earth at the part of the New Croton Dam under consideration and the great distance through the earth on the line of least resistance, the writer is of the opinion that the dam would have stood at this point without any core-wall, provided the up-stream part of the embankment of clayey material were carried down to join the hardpan, and that, with the core-wall as an added safeguard, this part of the dam would have had a greater factor of safety than the all-masonry section.

Vol. XXXII. FEBRUARY, 1906. No. 2.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

TEST OF A THREE-STAGE, DIRECT-CONNECTED CENTRIFUGAL PUMPING UNIT.

Discussion.*

By Elmo G. Harris, M. Am. Soc. C. E.

Mr. Harris. Elmo G. Harris, M. Am. Soc. C. E. (by letter).—It is a matter to be regretted that, under the pressure of severe competition, manufacturers of centrifugal pumps claim, and will often guarantee greater efficiencies than their machines can give; but there is no hope of bringing about the desired reform except by enforcing these guaranties. If this were commonly done, it would result in a twofold benefit: first, by preventing a mild degree of fraud on credulous purchasers; and, second, by ultimately improving the machines, until the efficiency is brought up to what the well informed believe to be economically attainable.

If the equivalent of the following clause were inserted in contracts for the purchase and installation of centrifugal pumps, the makers would be very conservative in their guaranties of efficiency, and, further, it would reveal to all parties concerned an economic principle too often neglected in fixing the cost of various installations:

"The contractor guarantees an efficiency of (   ) per cent., as determined by dividing the power delivered to the pump shaft by the power (due to the pump) in the water taken immediately after leaving that portion of the plant for which the contractor is re-

* Continued from December, 1905, *Proceedings*. See December, 1905, *Proceedings*, for paper on this subject by Philip E. Harroun, M. Am. Soc. C. E.

sponsible; and should this efficiency not be realized, the deficit, under normal working conditions, shall be estimated in horse powers, and, assuming the value of 1 h. p. to be ($) per annum, the present value of such a sum, paid annually for ( ) years, shall be estimated under a rate of interest of ( ) per cent. per annum, and this sum shall be deducted from the contract price of the pumping plant * * *." Mr. Harris.

If a corresponding bonus should be offered for exceeding a stated efficiency, the rapid improvement of centrifugal pumps would be assured. Much of the fault is due to the indifference or ignorance of the purchaser.

In regard to needed experimental knowledge, little is known about the losses caused by the friction of water gliding over metallic surfaces at such velocities as exist in centrifugal pumps. This should be found for revolving disks varying in diameter, surface finish, and speed. Considering the value of such data, the cost of the necessary apparatus would be small.

The answer to Mr. Richards' question, as to why efficiency should be insisted upon in centrifugal pumps and not in direct-acting reciprocating pumps, is that the cost of the power going into a centrifugal pump is very much greater than that going into a direct-acting pump; the latter is a cheap affair in first cost, taking steam direct from the boiler, while for a centrifugal pump there must be a rotating engine of some sort, and, if the pump is driven by electricity, there must be the prime motor (steam engine or water-wheel), the dynamo and the electric motor.

Noting Mr. Richards' allusion to a scheme for compressing air by combining air and water in a centrifugal machine, the writer would call attention to the Appendix* to the paper entitled, "Theory of Centrifugal Pumps and Fans," where such a scheme is somewhat clearly outlined.

Professor Le Conte was probably too modest to mention his excellent report of tests of centrifugal pumping plants published in the United States Report of Irrigation and Drainage Investigations, 1904. Such matter is scarce and valuable.

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* *Transactions*, Am. Soc. C. E., Vol. LI. p. 222.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the Volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

GABRIEL LEVERICH, M. Am. Soc. C. E.*

DIED NOVEMBER 28TH, 1905.

Gabriel Leverich was born on a farm about five miles from Elmira, New York, on August 19th, 1834. He was the son of Samuel and Sarah Leverich, for many years residents of New York City.

In his early years he attended what was then called the District (public) school in the vicinity, where he excelled in all his studies.

His taste seemed to be decidedly for mechanical work, and among his early inventions were a hay-rake, much like the one now generally used; a hay-fork for use by horse power, and other ingenious and practical inventions.

Two intimate friends and near neighbors, of about his own age, were graduates of the Rensselaer Polytechnic Institute, and this led him also to attend the institute, from which he was graduated in 1857.

His first engagement, after graduation, was at the Trenton Locomotive Works, where machinery for the manufacture of small arms was about to be introduced. In this connection he spent some time at the works near Springfield, Massachusetts, obtaining data from which he designed and constructed the necessary tools.

At this time Mr. Wiard was at Trenton, engaged in the construction of heavy ordnance embodying proposed improvements, and Mr. Leverich became interested in the matter. This, at a later date, led to his employment at Boston in the design of the "Thompson gun." The writer does not remember the details of this, but Mr. Leverich was one of the first to shrink on a jacket or insert a lining. This involved the refined accuracy of measurement with which we are now so familiar.

Other enterprises with which he was intimately connected were: the design and construction of apparatus for the destructive distillation of wood, in which all the products were saved; also, of machinery for the manufacture of fuel briquettes, by the compression of peat; and of improvements in the propulsion of tram cars.

Mr. Leverich was elected a Member of the American Society of Civil Engineers on July 6th, 1870, and from 1872 to 1877, inclusive,

*Memoir prepared by Francis Collingwood, M. Am. Soc. C. E.

served as Secretary of the Society. In these trying years of the Society's history, while nominally receiving a fair salary, he spent a considerable proportion of it in paying expenses which he deemed essential for its advancement, and for which the income was insufficient. Only his most intimate friends knew of this, however.

His services upon the first East River Bridge began when the approaches were under construction. He was responsible for the general features of the design of the Franklin Square Bridge, where the complexities of construction were very considerable. He took an important part, also, in the design of the New York station; and had entire charge of the changes, in both the New York and Brooklyn stations, made necessary by the great increase in traffic. These have more than doubled the carrying capacity of the bridge.

The rapid growth of traffic made it necessary also to improve the apparatus for propelling the cars on the bridge and increase its power. In carrying this out, Mr. Leverich showed an accurate knowledge of mechanical devices, and of the principles of mechanics; and the machinery is a model of ingenuity and effectiveness. This apparatus is described in a paper by him in Vol. XVIII of the *Transactions* of the Society. He contributed several other papers and discussions to the *Transactions*. All are noted for their clearness and accuracy of thought and expression. This was characteristic of all his work, and enabled him to answer all objections and carry his plans through to completion.

He died at the age of 71, having been an invalid for about six years, leaving a widow and a married daughter.

Mr. Leverich was bright and cheerful in disposition, and a good conversationalist. He was generous to a fault, and often, to his own detriment, helped others. He kept fully posted in all professional, industrial and political matters.

In early life he joined the Episcopal Church, and was always interested in its welfare.

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AMERICAN SOCIETY

OF

CIVIL ENGINEERS

March, 1906.

PROCEEDINGS - VOL. XXXII—No. 3

Published at the House of the Society, 220 West Fifty-seve
the Fourth Wednesday of each Month, except Ju

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL. XXXII. No. 3.

MARCH, 1906.

Edited by the Secretary, under the direction of the Committee on Publications.

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. ~96; died

NEW YORK 1906.

1, 1903; died

American Society of Civil Engineers.

OFFICERS FOR 1906.

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Term expires January, 1907:	*Term expires January, 1908:*
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ıse of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays, uly, Thanksgiving Day and Christmas Day.

F THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

ᴸLEPHONE NUMBER: - - - 533 Columbus.

ᴸE ADDRESS: - - - "Ceas, New York."

Vol. XXXII. MARCH, 1906. No. 3.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS.

CONTENTS:

MINUTES OF MEETINGS.

OF THE SOCIETY.

March 7th, 1906.—The meeting was called to order at 8.40 P. M.; Vice-President Kuichling in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 104 members and 18 guests.

The minutes of the meetings of February 7th, 14th and 21st, 1906, were approved as printed in the *Proceedings* for February, 1906.

A paper by Ernst F. Jonson, Assoc. M. Am. Soc. C. E., entitled "The Theory of Continuous Columns," was presented by the au

Ballots for membership were canvassed, and the follow ...96; died dates elected:

As Members. ...1, 1903; died

Philip Henry Coombs, Bangor, Me.
Frederick Leman Garlinghouse, Pittsburg.
John Francis Murray, Lancaster, Pa.

Emery Oliver, Oroville, Cal.
Arthur Breese Proal, Jr., New York City.
Edwin Charles Swezey, Brooklyn, N. Y.
Thomas Monroe Ward, Baltimore, Md.

As Associate Members.

Henry James Alexander, New York City.
Walter Loring Anthony, Providence, R. I.
Walter Buehler, Minneapolis, Minn.
Warren Vester Clark, Berkeley, Cal.
Vernon Royce Covell, Pittsburg, Pa.
Philip Henry Falter, Shawinigan Falls, Que., Canada.
Ramiro Ferradas, New York City.
John Peden Gardiner, New York City.
John Stanton Goodell, Cleburne, Tex.
Stephen Ford Holtzman, New York City.
Francis Rea Jones, Topeka, Kans.
Charles Whiteside Keith, Chicago, Ill.
Elsworth Mortimer Lee, New York City.
Richard Denny Parker, Houston, Tex.
George Loomis Robinson, New York City.
George Otis Sanford, Glendive, Mont.
Horace Richmond Thayer, South Bethlehem, Pa.
Jerome Frederick Wilhelm, Paragould, Ark.
Roger Butler Williams, Jr., New York City.

As Associates.

Daniel Griffith Ambler, Washington, D. C.
Harold Bouton, New York City.

The Assistant Secretary announced:

The transfer of the following candidates, by the Board of Direction, on March 6th, 1906:

From Associate Member to Member.

George Baum, Niagara Falls, Ont., Canada.
Arthur Lincoln Davis, New York City.
Alfred Courtney Lewerenz, Bremerton, Wash.
... Larkin Shaw, Chicago, Ill.
... Sparrow Wood, Providence, R. I.

From Associate to Member.

...renzo Abbott, New York City.

The election of the following candidates, by the Board of Direction:

As Juniors.

On February 6th, 1906:

Arthur Cassidy Everham, Detroit, Mich.
Norman Roosevelt McLure, Ardmore, Pa.
William Thomas Piper, Nashville, Tenn.

On March 6th, 1906:

Nora Stanton Blatch, New York City.
Howard Emory Bushnell, Hartford, Conn.
William Peyton Day, San Francisco, Cal.
Roger DeLand French, Worcester, Mass.
Elbert Allan Gibbs, Ithaca, N. Y.
Charles Edward Hayes, Cincinnati, Ohio.
Howard Benson Wilberforce Howie, Chattanooga, Tenn.
Arthur James McNeil, Whittier, Cal.
Vincent Roberts, New York City.

Adjourned.

March 21st, 1906.—The meeting was called to order at 8.40 P. M., Vice-President Kuichling in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 148 members and 28 guests.

A paper, by Theodore Cooper, M. Am. Soc. C. E., entitled "New Facts about Eye-Bars," was presented by the author.

The Assistant Secretary presented written communications on the subject from Messrs. Albert J. Himes, A. W. Carpenter and John Thomson, and the paper was discussed orally by Messrs. H. B. Seaman, Mansfield Merriman, Mace Moulton and the author.

The Assistant Secretary announced the following deaths:

John James Robertson Croes, Past-President, Am. Soc. C. E., elected Member, December 4th, 1867; President, January 16th, 1901, to January 15th, 1902; Vice-President, January 18th, 1888, to January 16th, 1889; Treasurer, November 7th, 1877, to January 18th, 1888, and Director, November 1st, 1876, to November 7th, 1877; died March 17th, 1906.

William Thomas Pierce, elected Member, May 6th, 1896; died March 10th, 1906.

Per Brynn, elected Associate Member, February 4th, 1903; died February 10th, 1906.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

March 6th, 1906.—President Stearns in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary, and present, also, Messrs. Bissell, Bowman, Ellis, Gowen, Green, Knap, Kuichling, Lewis, Noble, Schneider, Sherrerd, Swensson, and Webster.

The following committee was appointed to take charge of the arrangements for the Annual Convention:

Messrs. Charles S. Gowen, John W. Ellis, J. Waldo Smith, Morris R. Sherrerd, and Chas. Warren Hunt.

Applications were considered, and other routine business transacted.

Five Associate Members were transferred to the grade of Member, one Associate was transferred to the grade of Member, and nine candidates for Junior were elected.

Adjourned.

ANNOUNCEMENTS.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

MEETINGS.

Wednesday, April 4th, 1906.—8.30 P. M.—A regular business meeting will be held. Ballots for membership will be canvassed, and a paper, entitled "The Panama Canal," by A. G. Menocal, M. Am. Soc. C. E., will be presented for discussion.

This paper was printed in *Proceedings* for February, 1906.

Wednesday, April 18th, 1906.—8.30 P. M.—At this meeting a paper, entitled "A Complete Analysis of General Flexure in a Straight Bar of Uniform Cross-Section," by L. J. Johnson, M. Am. Soc. C. E., will be presented for discussion.

This paper was printed in *Proceedings* for February, 1906.

Wednesday, May 2d, 1906.—8.30 P. M.—A regular business meeting will be held. Ballots for membership will be canvassed, and a paper, entitled "The Control of Hydraulic Mining in California by the Federal Government," by William W. Harts, M. Am. Soc. C. E., will be presented for discussion.

This paper was printed in *Proceedings* for February, 1906.

Wednesday, May 16th, 1906.—8.30 P. M.—At this meeting a paper, entitled "The Scranton Tunnel of the Lackawanna and Wyoming Valley Railroad," by George B. Francis and W. F. Dennis, Members, Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings*.

ANNUAL CONVENTION.

The Thirty-eight Annual Convention of the Society will be held at The Hotel Frontenac, Thousand Islands, New York, on June 26th to 29th, 1906.

The general arrangements for the Convention are in the hands of the following Committee:

CHARLES S. GOWEN,
JOHN W. ELLIS, MORRIS R. SHERRERD,
J. WALDO SMITH, CHAS. WARREN HUNT.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all meetings:

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Society of Engineers, 17 Victoria Street, Westminster, S. W., England.

American Institute of Mining Engineers, 99 John Street, New York City.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Civil Engineers' Club of Cleveland, 1200 Scofield Building, Cleveland, Ohio.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Philadelphia, 1122 Girard Street, Philadelphia, Pa.

Engineers' Society of Western Pennsylvania, 410 Penn Avenue, Pittsburg, Pa.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Engineers' Club of Central Pennsylvania, Corner, Second and Walnut Streets, Harrisburg, Pa.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Teknisk Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Société des Ingénieurs Civils de France, 19 Rue Blanche, Paris, France.

Svenska Teknologföreningen, Brunkebergstorg 18, Stockholm, Sweden.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Sachsischer Ingenieur- und Architekten- Verein, Dresden, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Pacific Northwest Society of Engineers, 617–618 Pioneer Building, Seattle, Wash.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Memphis Engineering Society, Memphis, Tenn.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

The Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Cleveland Institute of Engineers, Middlesbrough, England.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

SEARCHES IN THE LIBRARY.

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling, compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

Copies of all lists of references are filed, so that in many cases it is only necessary to make a typewritten copy, which reduces the cost of searches to a minimum.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

ACCESSIONS TO THE LIBRARY.

From February 14th to March 13th, 1906.

DONATIONS.*

MANUAL OF EXAMINATIONS FOR ENGINEERING POSITIONS

In the Civil Service of the City of New York. Questions and Answers in 3 volumes. Vol. I, Part IV, Transitman and Computer; Vol. II, Part I, Assistant Engineer, Rapid Transit Commission. Paper, 9 × 6 in., illus., 2 parts. New York, The Engineering News Publishing Company, 1906. Vol. I, Part IV, 50 cents; Vol. II, Part I, 75 cents.

The preface states that in the "Previous Examination Papers" which have been included in this book, the questions may not, in all cases, be identical in wording with those actually given at the examinations, as copies of the original papers are not readily procurable, but they do embody the substance of the questions asked. In the section devoted to "Typical Questions and Answers," it is stated, that the answers indicate only in a general way what is required of the candidate, and are not intended to be perfect and complete, as reasonable variance of opinion may exist as to the best answer in many cases, owing to differences in interpretations of the question and in education and experience. Blank leaves have been inserted after the "Previous Examination Papers" to allow for the convenient addition of new sets, and the "Typical Questions and Answers" have been interleaved in order to provide space for notes, sketches and additions.

ELECTRIC POWER TRANSMISSION.

A Practical Treatise for Practical Men. By Louis Bell. Cloth, 9 × 6 in., illus., 721 pp. New York, McGraw Publishing Company, 1906. $4 net.

The preface states that, although there have been very few sensational changes in electric power transmission, the aggregate of minor changes in the past three years has made a new edition imperative. The author has found it needful to devote some special attention to the important accessory apparatus of which modern stations are full, and to make use of considerable new material of a more general sort, as well as to eliminate some descriptive matter which had to do with things which are obsolete and without historical importance. It is stated that in retaining merely as a matter of general interest a list of high voltage transmission plants nothing under 20 000 volts is included, as it has now become a hopeless task to keep track of 10 000 volt plants. There is an index of sixteen pages.

PRACTICAL ELECTRIC RAILWAY HAND BOOK.

By Albert B. Herrick. Second Edition, Revised and Corrected. Leather, 7 × 4 in., illus., 460 pp. New York, McGraw Publishing Company, 1906. $3 net.

The preface states that in this edition a number of sections have been rewritten and expanded, and new subjects have been introduced to accord with recent developments in the electric transportation industry. New methods of testing have also been described, and data on new types of apparatus have been added. It has been the author's effort to develop this Hand Book along the lines originally proposed and to keep within the limits of what is accepted as conservative engineering. He has also restricted the use of formulas and mathematics as far as possible, so as to make the text useful to the greatest number of co-laborers in this field. There is an index of nine pages.

MODERN TUNNEL PRACTICE.

Illustrated by Examples taken from Actual Recent Work in the United States and in Foreign Countries. By David McNeely Stauffer, M. Am. Soc. C. E. Cloth, 10 × 7 in., illus., 8 + 314 pp. New York, Engineering News Publishing Company, 1906. $5 net.

The preface states that, in the arrangement of this work, especial effort has been made to present modern practice in tunneling under as many varying conditions as possible, and to describe clearly and concisely the methods

*Unless otherwise specified, books in this list have been donated by the publisher.

actually adopted in carrying on the work under certain controlling conditions. The material used is very largely taken from the detailed descriptions found in the pages of technical journals and in the proceedings of engineering societies, supplemented by the personal experience of engineers and contractors. Illustration is very freely used. It is stated that the description of any especial method is prefaced by a brief statement of the physical conditions which called for some particular treatment. It is intended that the examples cited should be typical modern cases and cover a wide range of practice; no attempt has been made to include every important tunnel. What is said here on the subject of surveying is meant simply as a general statement of the processes involved and of their sequence. The composition, nature and use of modern explosives have been treated at considerable length; but much has been left out that was considered as having little bearing on tunnel building. There is a glossary of unusual terms used in tunneling and an index of six pages.

Gifts have also been received from the following:

Abner Doble Co. 4 pam.
Alabama—Geol. Surv. 1 map.
Am. Ceramic Soc. 1 vol.
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U. S. Geol. Surv. 6 vol., 99 maps.
U. S. Interstate Commerce Comm. 3 bound vol.
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U. S. Secy. of War. 4 bound vol.
Univ. of Pennsylvania. 1 vol.
Wellington, New Zealand—Harbour Board. 1 pam.
Western Australia—Govt. Rys. 1 vol.

BY PURCHASE.

The Irrigation Works of India. By Robert Burton Buckley. Second Edition. London, E. & F. N. Spon, Ltd.; N. Y., Spon & Chamberlain, 1905.

The Nature of Ore Deposits. By Richard Beck; Translated and Revised by Walter Harvey Weed. Vols. 1–2. New York and London, The Engineering and Mining Journal, 1905.

A Handbook of Rocks for Use Without the Microscope. By James Furman Kemp. Third Edition Revised. New York, D. Van Nostrand Company, 1906.

The Mineral Industry During 1904. Vol. XIII. New York and London. The Engineering and Mining Journal, 1905.

Die Entwickelung des Niederrheinisch-Westfälischen Stein-kohlen-Bergbaues in der zweiten Hälfte des 19 Jahrhunderts. Vol. VIII.—Disposition der Tagesanlagen, Dampferzeugung, Central-kondensation, Luftkompressoren, Elektrische Centralen. Berlin, Julius Springer, 1905.

SUMMARY OF ACCESSIONS.

February 14th to March 13th, 1906.

Donations (including 15 duplicates)	213
By purchase	6
Total	219

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ADDITIONS.

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BARNSLEY, GEORGE THOMAS. Chf. Road Engr., Allegheny County, Court House, Pittsburg (Res., Oakmont), Pa	Jun. Assoc. M. M.	May 31, 1892 May 7, 1894 Feb. 6, 1906
DAHM, SVERRE. Gen. Insp. of Designs, Rapid Transit R. R. Comm., 320 Broadway, New York City		Feb. 7, 1906
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NORTH, ARTHUR TAPPAN. Chf. Engr., National Fire Proofing Co. (Western Dept.), 806 Hartford Bldg., Chicago, Ill		Feb. 7, 1906
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WORMSER, MORITZ. 120 West 57th St., New York City	Feb. 6, 1906

DEATHS.

PIERCE, WILLIAM THOMAS. Elected Member, May 6th, 1896; died February 26th, 1906.

CROES, JOHN JAMES ROBERTSON. Past-President, elected Member, December 4th, 1867; died March 17th, 1906.

BRYNN, PER. Elected Associate Member, February 4th, 1903; died February 10th, 1906.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(February 10th to March 10th, 1906.)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Street Railway Journal*, New York City. Issues for first Saturday of each month 20c., other issues 10c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England, 25c.
(22) *Iron and Coal Trades Review*, London, England, 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Engineer*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, $1.
(29) *Journal*, Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l'Assoc. des Ing. Sortis des Ecoles Speciales de Gand*, Brussels, Belgium.
(32) *Memoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Genie Civil*, Paris, France.
(34) *Portefeuille Economique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Revue de Mecanique*, Paris, France.
(38) *Revue Generale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill., 10c.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussees*, Paris, France.
(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 8c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.

(54) *Transactions,* Am. Soc. C. E., New York City, $5.
(55) *Transactions,* Am. Soc. M. E., New York City, $10.
(56) *Transactions,* Am. Inst. Min. Engrs., New York City, $5.
(57) *Colliery Guardian,* London, England.
(58) *Proceedings,* Eng. Soc. W. Pa., 410 Penn Ave., Pittsburg, Pa., 50c.
(59) *Transactions,* Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering,* Indianapolis, Ind., 25c.
(61) *Proceedings,* Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World,* 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings,* Inst. C. E., London, England.
(64) *Power,* New York City, 20c.
(65) *Official Proceedings,* New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting,* London, England, 15c.
(67) *Cement and Engineering News,* Chicago, Ill., 25c.
(68) *Mining Journal,* London, England.
(70) *Engineering Review,* New York City, 10c.
(71) *Journal,* Iron and Steel Inst., London, England.
(72) *Street Railway Review,* Chicago, 30c.
(73) *Electrician,* London, England, 18c.
(74) *Transactions,* Inst. of Min. and Metal., London, England.
(75) *Proceedings,* Inst. of Mech. Engrs., London, England.
(76) *Brick,* Chicago, 10c.
(77) *Journal,* Inst. Elec. Engrs., London, England.
(78) *Beton und Eisen,* Vienna, Austria.
(79) *Forscherarbeiten,* Vienna, Austria.
(80) *Tonindustrie-Zeitung,* Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen,* Wiesbaden, Germany.
(82) *Dinglers Polytechnisches Journal.* Berlin, Germany.
(83) *Progressive Age,* New York City, 15c.

LIST OF ARTICLES.

Bridge.

Urban Bridges. Willis Whited. (58) Feb.
Girder Renewals, N. W. R., India.* G. H. List. (12) Serial beginning Feb. 9.
Secondary Members of the Island Span of the Blackwell's Island Bridge, New York.* (14) Feb. 10.
The Reinforced-Concrete Bridge at Trinidad, Col.* (14) Feb. 10.
Plate Girder Spans on the Chicago, Burlington & Quincy R. R.* (14) Feb. 17.
The Storage and Handling of Members for the Island Span, Blackwell's Island Bridge.* (14) Feb. 17.
Ferro-Concrete Viaduct at Gennevilliers, near Paris.* (11) Feb. 23.
Erection of Falsework and Pier Pedestals, Island Span, Blackwell's Island Bridge.* (14) Feb. 24.
The Pollasky Reinforced Concrete Bridge.* (14) Feb. 24.
Reconstruction of the Bismarck Bridge.* (14) Feb. 24.
The Elizabeth Eye-Bar Suspension Bridge.* L. Ramakers. (9) Mar.
The Danville Arch Bridge of the Cleveland, Cincinnati, Chicago & St. Louis Railway: A Description of a Double-Track Reinforced Concrete Bridge of Unusual Design.* (14) Mar. 3.
The New Portland Bridge.* H. A. Crafts. (14) Mar. 3.
Erection of the Upper Part of the Trusses of the Island Span of Blackwell's Island Bridge.* (14) Mar. 3.
Rapid Plate Girder Erection with a Derrick Car. (14) Mar. 3.
Construction Methods at the Stone Bridge at Hartford, Conn.* (14) Mar. 3.
Excavating and Concreting the New York Anchorage of the Manhattan Bridge.* (14) Mar. 3.
The Wabash River Bridge at Terre Haute, Indiana.* Malverd A. Howe, M. Am. Soc. C. E. (13) Mar. 8.
Calcul des Ponts Courbes. M. Résal. (43) 4e Trimestre, 1905.
Note sur un Système de Pont à Arc en Charpente et à Tirants Métalliques.* M. Thiollière. (43) 4e Trimestre, 1905.
Pont a Transbordeur sur le Port-Vieux à Marseille.* G. Leinekugel Le Cocq. (33) Serial beginning Feb. 24.
Gewölbegürten für Grosse Lasten.* Rudolph Heim. (78) Feb.

Electrical.

Researches on the Magnetic and Electric Properties of Various Kinds of Sheet Steel and Steel Castings. Gunnar Dillner and A. F. Enström. (71) Vol. 68.
Lightning Protection.* J. V. E. Titus. (Paper read before the Ohio Interurban Ry. Assoc. and the Cent. Elec. Ry. Assoc.) (72) Feb.
The Economy of Combined Railway and Lighting Plants. Ernest Gonzenbach. (Paper read before the Northwestern Elec. Assoc.) (72) Feb.

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*Illustrated.

Electrical—(Continued).

Repulsion Induction Motor.* Maurice Milch. (42) Feb.
Hydro-Electric Plant at Montereale.* (12) Feb. 9.
A Contribution to the Theory of the Single-Phase Induction Motor.* Val. A. Fynn. (26) Serial beginning Feb. 9.
The Chesterfield Electricity and Tramway Undertakings.* (26) Feb. 9.
Advantageous Use of Highly Magnetic Metal in Radiation Conductors. James Foster King. (27) Feb. 10.
Modern Underground Construction (for Electric Lighting and Power). W. D. Burford. (Abstract of Paper read before the Northwestern Elec. Assoc.) (27) Feb. 10.
The Cooper-Hewitt Single-Phase Converter.* (27) Feb. 10.
The Mansfield Electrical Undertaking.* (26) Feb. 16.
Series Transformers for Wattmeters. Lancelot W. Wild. (73) Feb. 16.
The Carbon Regulator for Automatic Booster Control.* (73) Feb. 16.
Mercury Arc Rectifier for Charging Storage Batteries.* (46) Feb. 17.
The Burlington, Vt., Municipal Electric Plant.* (27) Feb. 17.
The Design of an Isolated Power and Lighting Plant. (27) Feb. 17.
Electric Lighting at Tientsin.* (26) Feb. 23.
Phasemeters and Their Calibration. W. E. Sumpuer. (73) Feb. 23.
The Testing Room of the Worcester Electric Light Company.* (27) Feb. 24.
The District Supply System of the North Shore Electric Company, near Chicago.* (27) Feb. 24.
Canadian Niagara Development: The Remarkable Long Distance Electric Transmission for New York State.* Orrin E. Dunlap. (20) Mar. 1.
The Dutch Point Station of the Hartford Electric Light Company.* (27) Mar. 3.
Determination of Feeder-Drop. J. E. Wallace. (17) Mar. 3.
The Design of a Small Electric Power Station. James F. Hobart. (27) Serial beginning Mar. 3.
The Slide Rule as a Substitute for the Wire Table. Carl P. Nachod. (27) Mar. 3.
Central Station Operation and District Supply at Hillsboro, Ill.* (27) Mar. 3.
Residence Wiring.* Louis J. Auerbacher. (27) Mar. 3.
Electrical Conduit Work in Fireproof Buildings.* (27) Mar. 3.
L'Usine Hydro-Electrique d'Entraygnes et la Distribution d'Energie Electrique dans la Région de Toulon.* P. Caufourier. (33) Feb. 3.

Marine.

Analyses of the Trials of the Ferry-Boat *Scranton*. A. H. Potbury, E. A. Stevens, Jr., and O. von Voigtlander. (8) Jan.
Scotts' Shipbuilding and Engineering Works at Greenock.* (11) Feb. 9.
Excavation for Dry Dock No. 4, Brooklyn Navy Yard.* (14) Mar. 3.

Mechanical.

An Efficient Modern Steam Plant in Flour-Mill Service.* William H. Bryan. (1) Jan.
Some Notes on Fuel Briquetting in America. Clarence M. Barber. (1) Jan.
Report of a Test on a Portland Cement Plant.* E. C. Soper. (4) Feb.
The Prime Mover of the Future.* C. E. Sargent. (4) Feb.
Available Power and Cost of Operation of a Power Station for Waste Gases from a Blast Furnace Plant. H. Freyn. (4) Feb.
Power Required by Machine Tools, with Special Reference to Individual Motor Drive.* G. M. Campbell. (58) Feb.
Tests of Small Compressors.* Max Kurth. (45) Feb.
The Flow of Steam Through Nozzles.* (11) Feb. 2.
Superheated Steam. Michael Longridge, M. Inst. C. E. (Lecture delivered before the Bradford Eng. Soc.) (11) Feb. 2.
Gas-Works Construction in Canada: New Coal-Gas Plant at the Winnipeg Gas-Works.* (66) Feb. 6.
Design and Construction of Steam Turbines. Frank Foster. (47) Feb. 10.
A Large Testing Machine.* (47) Feb. 10.
A Modern Retort-House Equipped for Horizontal Working.* (66) Feb. 13.
The Steam Consumption of Reciprocating Engines. T. Stevens and H. M. Hobart. (27) Feb. 17.
Improved Westinghouse Steam Turbine.* (47) Feb. 17.
Weight in Winding Drums.* R. H. Collingham. (12) Feb. 23.
The Economy of Steam Turbines Compared with that of Reciprocating Engines. T. Stevens and H. M. Hobart. (27) Feb. 24.
Mechanical Plant of the New Wanamaker Store in New York.* (14) Serial beginning Feb. 24.
Vertical Retorts for the Production of Illuminating Gas.* William Young. (66) Feb. 27.
Smoke Prevention in the Modern Power Station. John B. C. Kershaw. (64) Mar.
Boiler-House Economy.* Alfred W. Bennis, M. I. Mech. E. (10) Mar.

*Illustrated.

Mechanical—(Continued).

Electricity in the Foundry.* H. S. Knowlton. (10) Mar.
The Conditions of Mechanical Draught Production. Walter B. Snow. (10) Mar.
The Suction Gas Producer.* W. H. Booth. (10) Mar.
Some Features of Modern European Gas Holder Design.* (13) Mar. 1.
The East Jersey Pipe Co.'s Lock-Bar Pipe Plant. (14) Mar. 3.
Boiler Efficiency Tests. George T. Hanchett. (27) Mar. 3.
Gas Producers for Power. Julius Wile. (From Paper read before the Technology Club.) (20) Mar. 8.
Molding Sands. H. E. Field. (62) Mar. 8.
Manufacture of Hydraulic Cements. L. L. Stone. (Paper read before the Univ. of Mich. Eng. Soc.) (19) Mar. 10.
Fabrication des Briques Silico-Calcaires par les Procédés Röhrig et König.* (33) Jan. 27.
Verdampfungsversuche an einem Wasserrohrkessel System "Gehre."* (Tr. from the Russian by Richard Starck.) (52) Serial beginning Jan.
Gasofen und Halbgasofen. W. Tafel. (50) Feb.
Gusseiserne Muffenrohrverbindungen.* Gustav Simon. (50) Feb.
Die Dessauer Vertikalretorte.* J. Bueb. (48) Feb. 10.
Neue Orsat-Apparate für die Technische Gasanalyse.* C. Hahn. (48) Feb. 10.
Die Regelung Mehrstufiger Dampfturbinen.* Harry Jansson. (48) Feb. 10.
Ueber die Bildung von Hohlräumen in Stahlblöcken und die Mittel zu Ihrer Verhinderung.* J. Riemer. (50) Feb. 15.
Die Blechwalzwerks-Anlagen der Central Iron and Steel Company, Harrisburg, Pa.* Oskar Simmersbach. (50) Feb. 15.
Ueber das Formen der Stahlwerkskokillen und Deren Haltbarkeit. A. Messerschmitt. (50) Serial beginning Feb. 15.
Untersuchungen Explosibler Leuchtgas-Luftgemische.* F. Häusser. (48) Feb. 17.
Beitrag zur Frag: Kann Ueberhitzter Dampf Wasser Enthalten?* Fritz L. Richter. (48) Feb. 24.

Metallurgical.

The Thermal Transformations of Carbon Steels.* John Oliver Arnold and Andrew McWilliam. (71) Vol. 68.
Overheated Steel.* Arthur Windsor Richards and John Edward Stead. (71) Vol. 68.
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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE SCRANTON TUNNEL OF THE LACKAWANNA AND WYOMING VALLEY RAILROAD.*

By George B. Francis and W. F. Dennis, Members, Am. Soc. C. E.

To be Presented May 16th, 1906.

During the years 1901, 1902 and 1903, Westinghouse, Church, Kerr and Company constructed a double-track, high-speed, electric railroad, on private right of way, in the heart of the Northern Anthracite coal field, with termini at Wilkes-Barre on the south, and Scranton on the north, a distance of about 20 miles.

The entrance to Scranton was over the hill on the southerly side of the city, with 4% grades, on a temporary line built to permit an early opening of the road.

During 1904 and 1905 a tunnel was constructed through this hill, on a permanent line and at moderate grades. The object of this paper is to describe the construction of this tunnel, which was accomplished in record time, and which contained some interesting engineering features.

The preliminary surveys for the tunnel line were made in 1902, and the first location was under Irving Avenue, Scranton, one block nearer the center of the city than the existing line, the object being

* That portion of the paper describing the tunnel generally has been prepared by Mr. Francis, Civil Engineer for Westinghouse, Church, Kerr & Company, and that portion describing the contractor's plant and operations has been prepared by Mr. Dennis, Vice-President of the Rinehart & Dennis Company, General Contractors.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

PLATE XVI.
PAPERS, AM. SOC. C. E.
MARCH, 1906.
FRANCIS AND DENNIS ON
TUNNEL CONSTRUCTION.

to take advantage of the extensive old mine workings, known as the Dunmore No. 3 coal vein, cropping out just beyond the company's power plant. It was hoped that this might prove an easy, quick and inexpensive means of tunneling, but surveys in the mines demonstrated that the workings were not favorable to the accomplishment of this plan. The drift was above and below the tunnel grade and crossed the profile twice, introducing puzzling questions of support.

It was finally determined to locate a line which would avoid these old mine workings and clear the limits of the coal yet to be mined. The new line was located to the south of Irving Avenue, under private land contiguous to Crown Avenue.

The contract for a single-track tunnel was executed on June 1st, 1904, with the Rinehart and Dennis Company, of Washington, D. C. Time was the essence of this contract, which provided for the completion of the work, to a stage admitting of track laying throughout, in a period of 16 months, under a penalty and bonus clause providing for the payment of $200 per day.

Work was commenced on July 5th, 1904, after a brief delay on account of right-of-way matters. The first round of holes for the heading was fired on August 12th, 1904, at the north portal, about one month being consumed in excavating the approach cut. The excavation was completed and the tunnel ready for laying track on July 18th, 1905, the contractors earning a bonus of $19 000.

From a knowledge of the geology in the vicinity, and a determination of the nature of the rock through which the tunnel would be driven, it was evident that some portions would require lining; but, at the time the work was let, it was not known how much.

Typical sections were prepared showing the method of treatment of expected conditions.

After the work had progressed for a period of six months it was quite evident that nearly all the tunnel was in rock of inferior quality, and would require lining of some character for nearly its entire length. The rock had a tendency to slab off or drop in small pieces rather than settle down on the supports in large masses.

To effect a saving in the initial cost of construction, it was determined to introduce some form of permanent timber lining which would take the place of the more expensive masonry section. Instead of placing the timber outside of a future masonry section, it was put in the position of the masonry section, thus effecting a con-

siderable saving, both in excavation and masonry. At such future time as it is desired to reline the tunnel, short stretches of the timber may be removed and replaced with timber or masonry.

The dimensions of the tunnel are as follows:

Total length, face to face of portal masonry.	4 747	ft.
Total length excavated as tunnel.........	4 640	"
Portal masonry extended at north end....	16	"
*Portal masonry extended at south end....	91	"
Width, in clear, inside...................	17	"
Height above top of rail................	20	"
Depth below top of rail.................	2	"
Arch, semi-circular, radius..............	8.5	"
Shaft No. 1, 10 by 20 ft., depth..........	104	"
Shaft No. 2, 10 by 20 ft., depth..........	180	"

Surveying.—The alignment of the Scranton Tunnel presented some of the difficulties usually attendant on work of this character; the principal ones were:

1.—The center line of the tunnel at the surface passed through a great many buildings;

2.—There was no higher ground in the vicinity on the axis of tunnel;

3.—The surface was quite uneven, and five permanent points had to be established on the prime base line on Crown Avenue;

4.—Owing to the configuration of the surface at the portals, curves had to be extended into each end of the tunnel;

5.—The shafts were located to one side of the center line of the tunnel, and came partly without and partly within the tunnel prism.

A prime base was established along Crown Avenue, 45 ft. from, and parallel with, the projected tunnel line. This line was run and re-run, reversing the transit telescope, differences were eliminated and adjusted, and permanent points established. Then, as a check on the work, a line was run over the work several times, using no back-sights.

After a base line with no apparent error had been established in Crown Avenue, tangents were brought ahead from the existing tracks at each end to an intersection with this base line; and, at

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* Extended to carry the roadway over the tunnel.

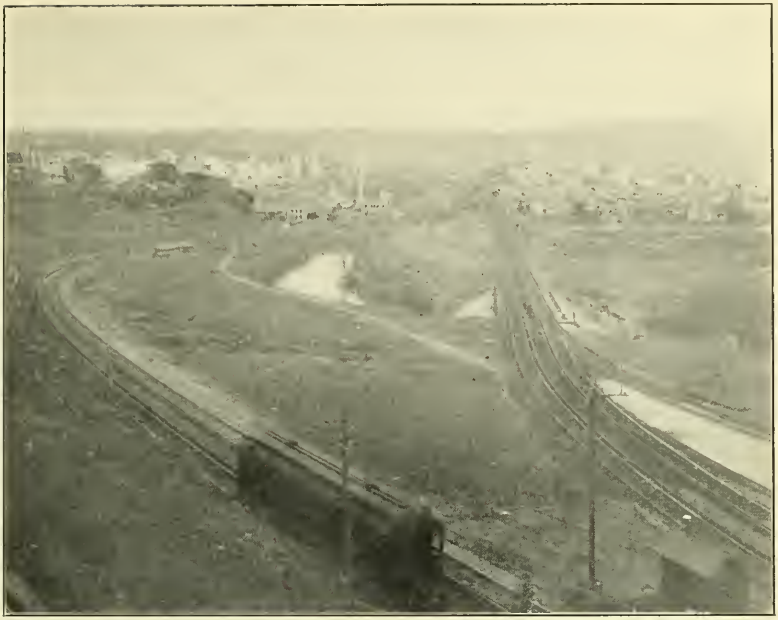


Fig. 1.—North Approach, Scranton Tunnel. Tunnel Line on Left, Temporary Line on Right.

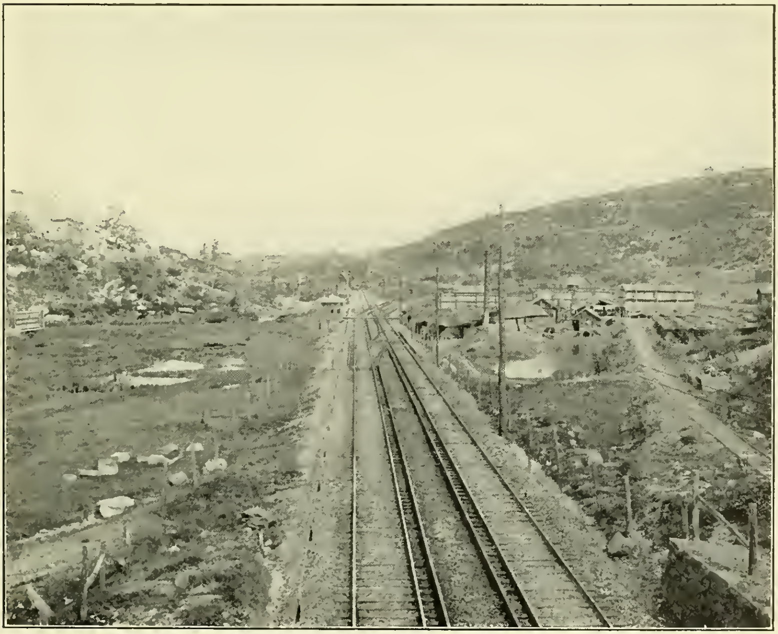


Fig. 2.—South Approach, Scranton Tunnel.

the north end, a base 80 ft. long was obtained for measuring angles. Angles were turned a sufficient number of times to insure their correct valuation, and curves were run in and checked by direct measurements from tangents and by complete triangles, in which all distances and all angles were measured. These triangles were remeasured and adjusted until no appreciable error remained. By this method a permanent point was established on the center line of the curve at a convenient place near each portal.

From the points established along Crown Avenue, other points convenient to the shafts were put in place, and from these and at right angles thereto, monuments were established about 51 ft. away to take the line down the shafts. As the center line of the tunnel was 45 ft. distant, another offset was necessary at the foot of the shafts. It was complicated still further at these points, in the early stages of the work, by the presence of pillars or shoulders of rock left on either side at the bottom of the shafts for the protection of the hoisting apparatus, and two more offsets had to be made to get to the center line. After the removal of this rock the line was taken directly from the wires in the shafts by producing the lines about 200 ft. either way and repeating the operations until there was no appreciable error.

The method of taking the line into the shafts, as shown in Fig. 1, was simple, but great care was taken to make the lines accurate. An observer with a transit, with a fixed fore-sight, was always left on the surface of the ground. Plumb-bobs, weighing about 30 lb., were suspended with copper wires from points about 2 ft. above the curb, or top, of the lining of the shaft.

The wires were hung over a notched bolt, running through a U-shaped piece of iron, with a thumb-screw and spring adjustment. The plumb-bobs were steadied in casks of water or oil at the bottom of the shafts, and the casks had covers so that dripping water would not disturb the surface of the liquid. Even with this precaution, there was always some tremor in the wires, owing to the fact that they were frequently struck by falling water.

The lines met in the headings between the south portal and Shaft No. 1 with a variation of 0.16 ft.; between Shafts Nos. 1 and 2 with a variation of 0.02 ft.; and between Shaft No. 2 and the north portal with a variation of 0.23 ft. In no case was the center line, as run from the shafts, more than $1\frac{7}{8}$ in. from the actual center line.

The grades met between the south portal and Shaft No. 1 with a variation of 0.02 ft.; between Shafts Nos. 1 and 2 with a variation of 0.07 ft.; and between Shaft No. 2 and the north portal with a variation of 0.05 ft.

Alignment and Grade.—At the south end of the tunnel a 4° 14′ curve, with a spiral 218.75 ft. long, was run in to connect the center line of the existing track with the center line of the tunnel. All the spiral and 270.34 ft. of regular 4° 14′ curve was inside the tunnel portal, or a total of 489.09 ft. of curve, was run in at this end.

At the north end, owing to a very narrow valley, a 10° curve, with spirals 400 ft. long, at each end, had to be introduced. One spiral, together with 186.21 ft. of regular 10° curve, or a total of 586.21 ft. of curve, was inside the tunnel portal.

At this end, as at the south portal, lines were given as the work progressed, so that the excavation for the heading was taken out to the full tunnel section.

The tunnel, generally, has a grade of 1%, but, for a short distance, at the north end it is 0.8 per cent.

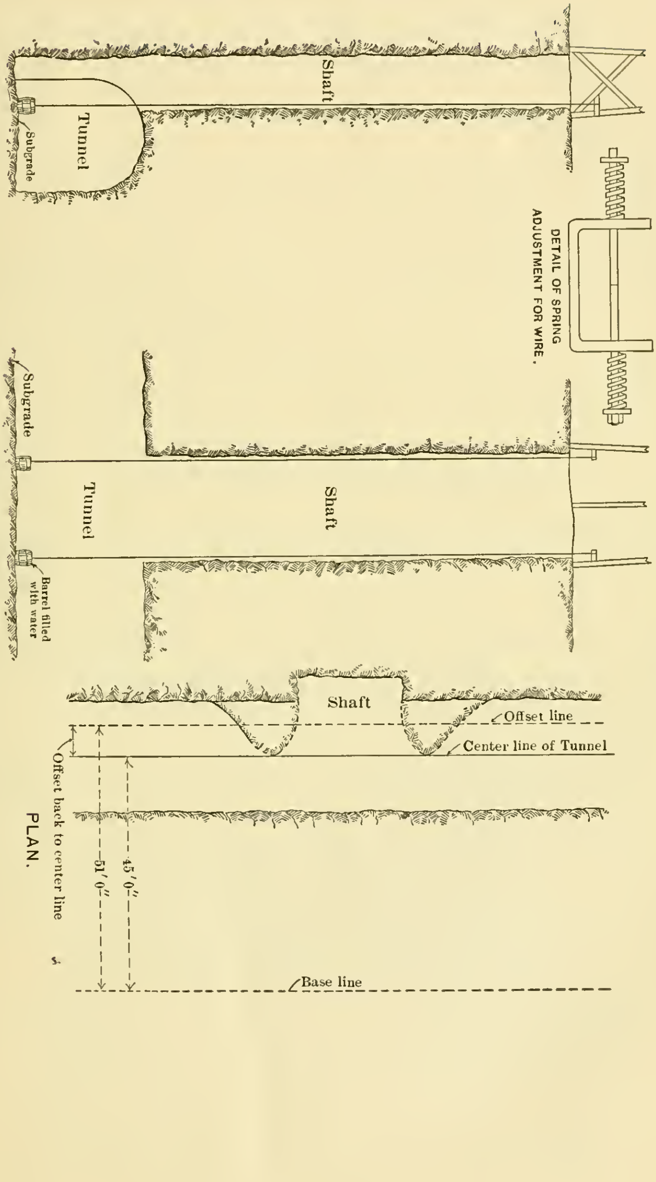
Excavation.—The excavation was carried on at six different points of attack, two shafts being sunk, about 1 500 ft. apart, from which headings were worked in both directions, as well as at the north and south portals. Fig. 2 shows that 56%, or more than one-half, the tunnel excavation was removed through these shafts, the work being expedited thereby fully a year in time.

The method of tunneling was in accordance with the usual American practice. The top heading was taken out to the full size of the section, 9 by 21 ft., and the length carried on in advance of the bench varied from 50 to 800 ft. The bench was afterward split in two lifts.

The quantity of excavation per linear foot of tunnel was as follows:

	Rock Section. Cu. Yd.	Masonry Section. Cu. Yd.	Timber Section. Cu. Yd.
Headings	4.71	7.04	8.38
Benches	9.00	11.00	12.00
Totals	13.71	18.04	20.38
Falling beyond payment line	1.00	1.13	1.18

METHOD OF TRANSFERRING THE CENTER LINE DOWN A SHAFT.



SECTION.

ELEVATION.

Fig. 1.

Disposal of Spoil.—The portal excavation at the north end was dumped along the bank of Roaring Brook. At the south end the greater portion was loaded on cars and used as rip-rap at a point on the line of the road nearly 16 miles away.

The excavation from Shaft No. 1 was dumped on low land near by, purchased for the purpose; and that from Shaft No. 2 was hauled by surface incline to a dump about 1 200 ft. away and from 50 to 60 ft. higher.

The work was carried on by night and day in two 10-hr. shifts.

Lining.—A small quantity of temporary lining, outside of the masonry section, was required at each portal and at Shaft No. 1, at which points there was danger of immediate and frequent falls from the roof. At other places these falls were less frequent, but the roof would slab off to such an extent as to make it necessary to line, either with timber or masonry, before beginning the operation of regular train service.

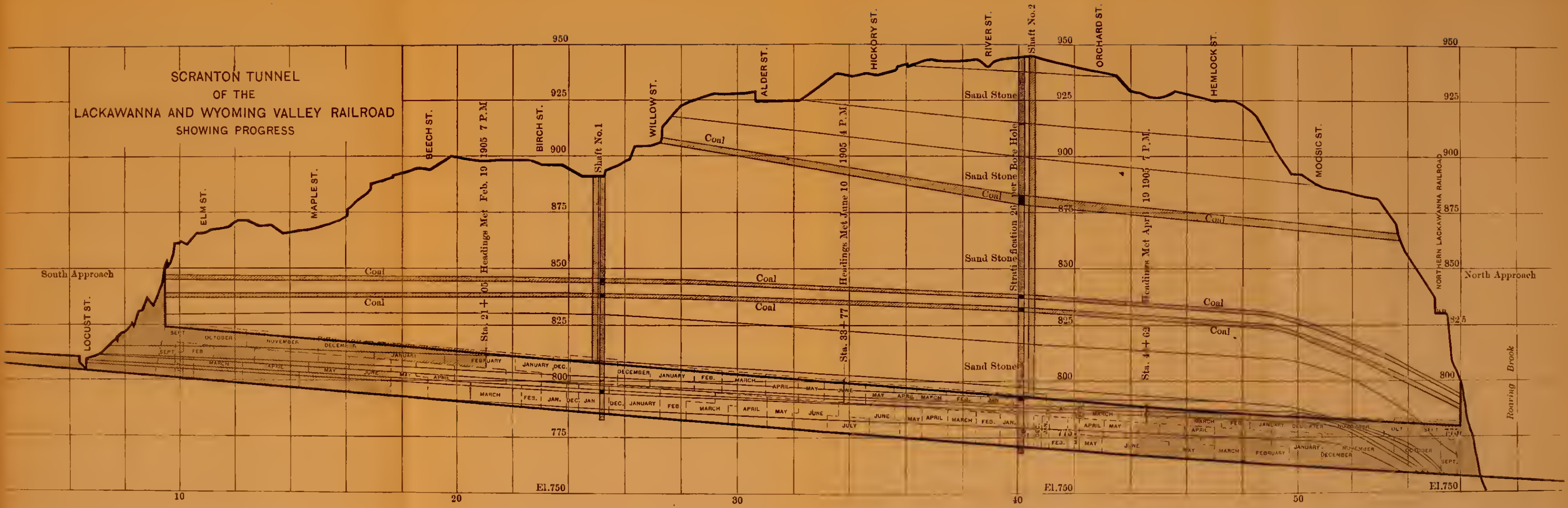
Concrete masonry lining was adopted for those portions of the tunnel near the portals and near the shafts, where the rock was of the poorest quality. Lining of this kind, being the most expensive, was restricted as much as possible, and permanent timber lining substituted.

The permanent timber lining is of the usual pattern of voussoir ribs, 5 ft. on centers, with a 4-in. lagging. The yellow pine timber used for this purpose should last about 12 years before requiring renewal, and at the end of that time it can be renewed or replaced with masonry.

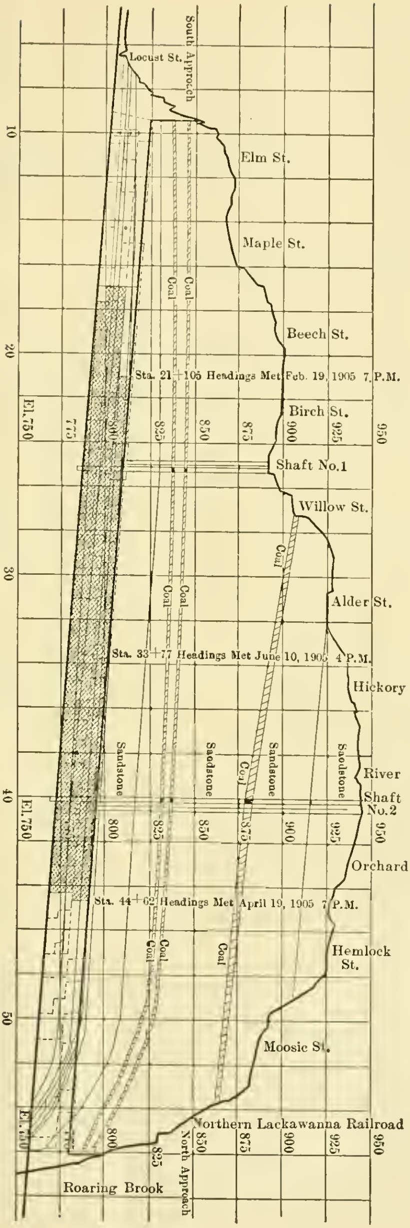
At three different points in the tunnel there were a few hundred feet where the rock was found sufficiently good to require no lining. There has been one fall, of moderate size, however, since the opening of the tunnel for regular traffic. These portions of the roof are being carefully watched, and if there is much uncertainty as to falls it will have to be lined.

The following are the lengths of the different sections:

Plain rock section........................	1 305 ft.
Timber-lined section......................	2 717 "
Masonry section...........................	725 "
Total.....................................	4 747 "


SCRANTON TUNNEL
OF THE
LACKAWANNA AND WYOMING VALLEY RAILROAD
SHOWING PROGRESS
South Approach
North Approach
LOCUST ST.
ELM ST.
MAPLE ST.
BEECH ST.
BIRCH ST.
Shaft No.1
WILLOW ST.
ALDER ST.
HICKORY ST.
RIVER ST.
Shaft No.2
ORCHARD ST.
HEMLOCK ST.
MOOSIC ST.
NORTHERN LACKAWANNA RAILROAD
Roaring Brook
Sta. 21 + 05 Headings Met Feb. 19 1905 7 P.M.
Sta. 33 + 77 Headings Met June 10 1905 4 P.M.
Sta. 44 + 63 Headings Met April 19 1905 7 P.M.
Stratification 26 per Bore Hole
Coal
Sand Stone
950
925
900
875
850
825
800
775
El.750
10
20
30
40
50

SCRANTON TUNNEL: PROPORTION OF MATERIAL TAKEN OUT AT SHAFTS AND PORTALS.



Shaft No.1 ..... = 29 265 cu. yd.
" " 2 ..... = 20 090 " "
Total by shafts = 49 355 " "
" " portals = 38 105 " "
" Excavation = 87 460 " "
56% taken out through shafts.

FIG. 2.

The proportions of concrete mixture for the two classes specified were as follows:

Class "A" concrete, in the arches of the roof or in the side walls where the thickness does not exceed 26 in., 1 part cement, 2 parts sand and 4 parts broken stone.

Class "B" concrete, in the side walls or tunnel arches, where the backing is rock in place, 1 part cement, 3 parts sand and 5 parts broken stone.

Table 1 gives the cement tests made.

TABLE 1.—AVERAGE TENSILE STRENGTH OF BRIQUETTES TESTED.
Lehigh Portland Cement.

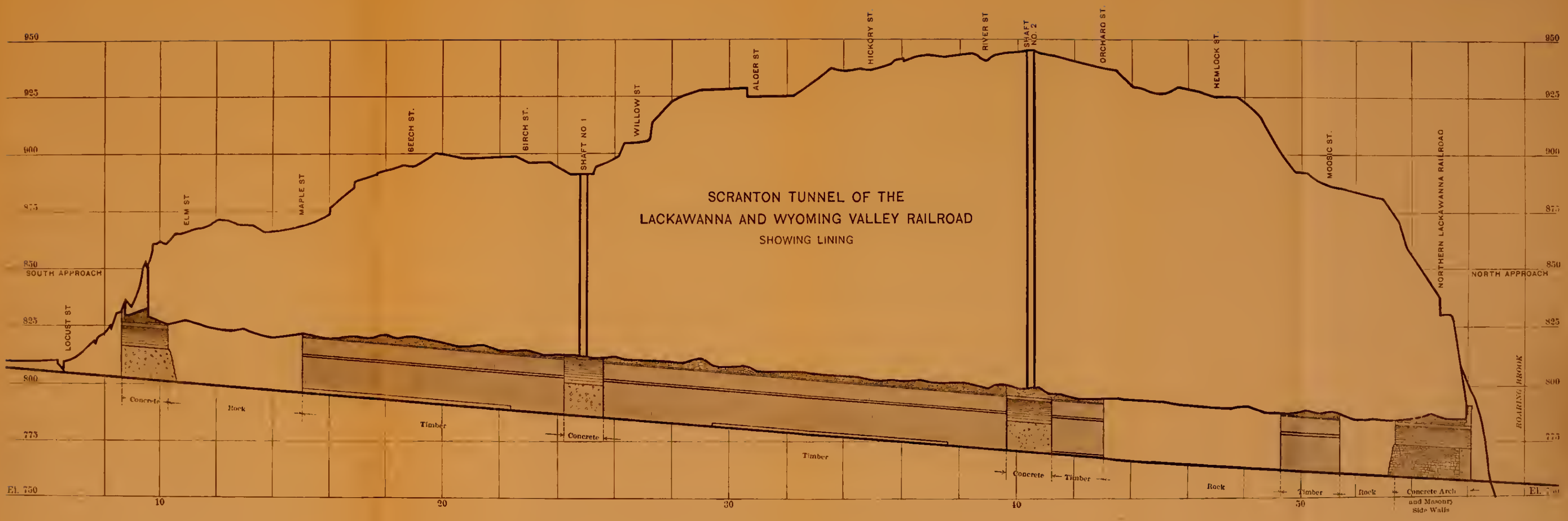
Consignment.		Briquettes made.	Fineness.		Boiling Test.	Mixture.				
						Neat, 20% Water.			1 Cement, 3 Sand, 10% Water.	
Car No.	No. of Barrels.		100 mesh.	200 mesh.		1 day.	7 days.	28 days.	7 days.	28 days.
L. V. 68 119......	150	6	....	....	......	......	......	......	303	400
L. V. 5 379.......	150	18	89	72	N. G.	293	709	857	......	
L. V. 71 827......	150	30	87	69	O. K.	211	687	795	267	......
L. V. 64 005......	150	30	90	75	O. K.	269	739	800	337	418
L. V. 70 298......	150	30	90	73	O. K.	168	765	842	246	487
L. V. 64 198......	150	30	90	73	O. K.	271	724	851	250	393
L. V. 61 662......	150	30	90	72	......	264	687	796	234	337
D. L. & W. 33 082	150	30	90	72	O. K.	301	742	854	300	330
D. L. & W. 26 354	150	30	90	70	O. K.	383	849	937	244	434
										373

NOTE.—With the exception of the first item, the number of briquettes tested in each instance was six, and the figures given herein are the averages of the various tests.

Shafts.—There are two shafts (10 by 20 ft. neat size), which were located at available points, and divided the tunnel into three nearly equal parts. Shaft No. 1 has a depth of 104 ft. and Shaft No. 2 a depth of 180 ft.

These shafts were placed to one side of the center line of the tunnel, the object being to provide a place where the shaft leads and rigging would be out of the way and be less likely to be injured by blasting, thus obviating the resultant delays in the early prosecution of the work at these points. This location of the shafts was also a safeguard against possible accidents caused by dropping tools, etc., from above, or spoil from the cars while being hoisted. As a further protection to this hoisting apparatus against blasting,

PLATE XIX.
PAPERS, AM. SOC. C. E.
MARCH, 1906.
FRANCIS AND DENNIS ON
TUNNEL CONSTRUCTION.



two shoulders were left on either side of the shafts, and this rock was not removed until the tunnel excavation had advanced to points where the shaft rigging was safe from the effects of blasting.

This arrangement of the shafts was specially requested by the contractor, as it would enable him to lay the tracks in the tunnel in such a manner that the cars loaded with spoil, as they arrived at the mouth of the shafts, could, if found necessary, be shifted into either of the cages and raised to the surface without loss of time, thus securing the maximum service from this part of the equipment. In the actual prosecution of the work, however, this was not found necessary.

Each shaft was divided into two compartments by center cross-bracing, and these compartments formed the well for the cage or elevator, which was 8 ft. 7 in. square. The cages were built up of steel shapes, strongly riveted together, with a top cover of plate iron, the only wood about them being the floors. Channel guides were provided for the entire height of the cages, about 9 ft., and these engaged 5 by 8-in., dressed, hard-pine leads firmly attached to the sides of the shafts.

From each cage a wire hoisting cable, $\frac{7}{8}$ in. in diameter, was attached to the engine, both cables winding on the same drum, the load on the engine being lightened to the extent of the empty cage descending, which acted as a counterweight. As a safety appliance, the cages were directly cross-connected with a $\frac{3}{4}$-in. cable operating over separate sheaves, which acted as a direct counterweight. The hoisting engines were horizontal, two-cylinder, single-drum, steam-driven, and of 40 h. p.

The head-house, or hoisting frame, consisted of three framed bents, of 12 by 12-in. and 12 by 14-in. hard-pine timber, well tied together at top and bottom. Between this hoisting frame and the engine a guide with two sheaves was set up to keep the cable at the proper angle to feed to the drum.

One man was always stationed at the top and another at the bottom of the shafts, and no movement of the cages was permitted until they had interchanged signals, and the man at the top of the shaft had given the signal to the engineer. No accident resulted from the use of the shafts and cages.

The two shafts have been left for ventilating purposes; the rock

sides, being fairly sound, will not cave in to any extent, even if the timber lining, which has been left in place, decays.

The openings have been covered with crib houses, about 10 ft. high, and built of 8 by 8-in. second-hand timber, laid horizontally, and solidly drift-bolted. A flat roof-grating, of the same material, completes the structure, and provides a covering of sufficient strength to prevent anyone, especially children, from tearing off the boarding and getting into trouble by falling into the shaft. This method of treatment was much less expensive than building masonry structures.

General Progress.—The general progress is shown by the following:

Contract signed..................	June	1st, 1904.
Work started at Shaft No. 1......	July	25th, 1904.
" " " " " 2.....	August	2d, 1904.
" " " South Portal....	July	11th, 1904.
" " " North Portal....	July	13th, 1904.
First round holes for heading fired at north end..................	August	12th, 1904.
First round holes for heading fired at south end..................	September	8th, 1904.
Excavation entirely completed....	July	18th, 1905.
Meeting of headings between South Portal and Shaft No. 1..	February	19th, 1905.
Meeting of headings between North Portal and Shaft No. 2..	April	19th, 1905.
Meeting of headings between shafts	June	10th, 1905.

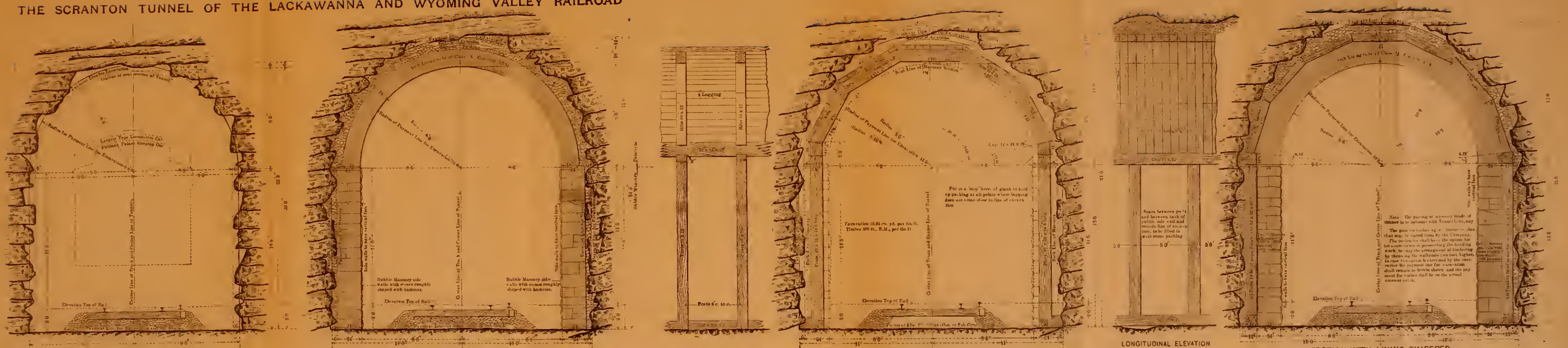
The progress is shown by the profile, Plate XVIII.

Maximum Progress.—The maximum progress was as follows:

Maximum rate per month for all headings..	881.5 ft.
" " " " " " benches ..	745.0 "
" " " week " " headings..	237.0 "
" " " " " " benches ..	256.0 "
" " " month of any one heading	261.0 "
Maximum rate per week of any one bench..	85.0 "
" " " month for shafts........	111.0 "

PLATE XX
PAPERS, AM. SOC. C. E.
MARCH, 1906.
FRANCIS AND DENNIS ON
TUNNEL CONSTRUCTION.

THE SCRANTON TUNNEL OF THE LACKAWANNA AND WYOMING VALLEY RAILROAD



SECTION WITHOUT LINING

SECTION WITH LINING

SECTION WITH LINING

LONGITUDINAL ELEVATION OF TIMBERING

SECTION WITH LINING TIMBERED

Table 2 shows the progress in detail.

Lighting.—The tunnel is lighted by electricity, 16-c. p. incandescent lamps being installed at intervals of approximately 40 ft. throughout. The lamps are arranged in groups, of six 110-volt lamps per group, all lamps of each group being in series between the third rail and the track rail, and are located as high as possible on one side of the tunnel walls. Every sixth lamp is connected to the rail circuit with a No. 4 Brown and Sharp bare copper wire, which is securely fastened to the nearest tie and to the side wall of the tunnel in such a manner as to be entirely clear of moving equipment and safe from disturbance by the track men.

There is one feeder wire, of the same polarity as the third rail, running from end to end of the tunnel, to which each group of lights is tapped at every sixth lamp. The connection from the last lamp of each group is run directly to the track.

The positive feeder is connected to the third rail through a single-pole, 500-volt, 25-ampere, quick-break, knife-switch, and one 25-ampere, open fuse, mounted on a 50-ampere D. & W. fuse-block, all enclosed in a water-proof, wooden box, at the north portal, by which control is had of all the lamps in the tunnel from a single convenient point.

A 500-volt, 3-ampere, porcelain, open-fuse cut-out, of D. & W. make, is placed in each tap between the feeder and the first lamp. This affords protection to each individual circuit, so that an accident, which would cause a short circuit of any one of the independent groups, would not throw the entire installation out of commission.

The cost of the installation was about $8 per light.

Track.—The double track, approaching the tunnel at either end, is gauntleted through the tunnel, and two third rails are provided for electric contact, thus making six rails in a single-track tunnel.

The track is of 90-lb., Am. Soc. C. E. section, for the running rail, laid with tie-plates; and of 75-lb., Am. Soc. C. E. section for the third rail, on standard ties and rock ballast, 1 ft. deep under the ties, all rails being properly bonded for the return circuit.

Signals.—The Union Switch and Signal Company's electric train-staff system of block signaling is in use, for the prevention of

TABLE 2.—MONTHLY PROGRESS ON SCRANTON TUNNEL, IN LINEAR FEET.

DATE.	HEADINGS.						BENCH.						TOTALS.		Remarks.
		Shaft No. 1.		Shaft No. 2.				Shaft No. 1.		Shaft No 2.					
Month of	South Portal.	South.	North.	South.	North.	North Portal.	South Portal.	South.	North.	South.	North.	North Portal.	Heading.	Bench.	
July, 1904	Started July 11.	......	......	......	......	Started July 13.	......	......	......	......	......	......	......	......	9 + 56 to 55 + 96.
August	......	......	......	......	......	41	......	......	......	......	......	......	41	......	First round of holes fired Aug. 12.
September	88	......	......	......	......	135	24	......	......	......	......	69	223	93	
October	194	75	63	33	30	104	30	10	10	......	......	131	499	181	
November	261	13	17	11	26	216	......	46	40	10	10	196	544	302	
December	201	59	116	39	16½	130	......	47	56	29	31	110	561½	273	
January, 1905	254	140	156	97	92½	142	......	85	122	71	26	120	881½	424	
February	151	102	96	125	25	100	40	73	95	84	64	160	599	516	
	Met Fe	b. 19th													
March	......	......	158	103	126	124	175	164	149	105	45	107	511	745	
April	......	......	114	82	101	142	202	174	136	89	40	50	439	691	
					Met A	pr. 19th									
May	......	......	93	103	......	......	183	131	94	78	42	120	196	649	
June	......	......	70	75	......	......	100	27	124	106	87	143	145	587	
			Met Ju	ne 10th											
July	......	......	......	......	......	......	......	27	80	72	......	......	......	179	
							754	784	906	544	346	1 206	4 640	4 640	

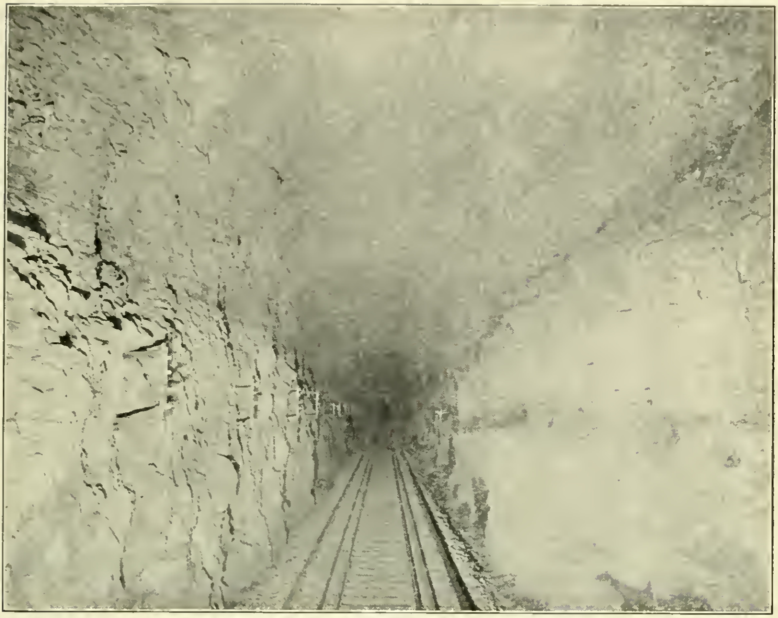


FIG. 1.—SCRANTON TUNNEL; PLAIN ROCK SECTION.

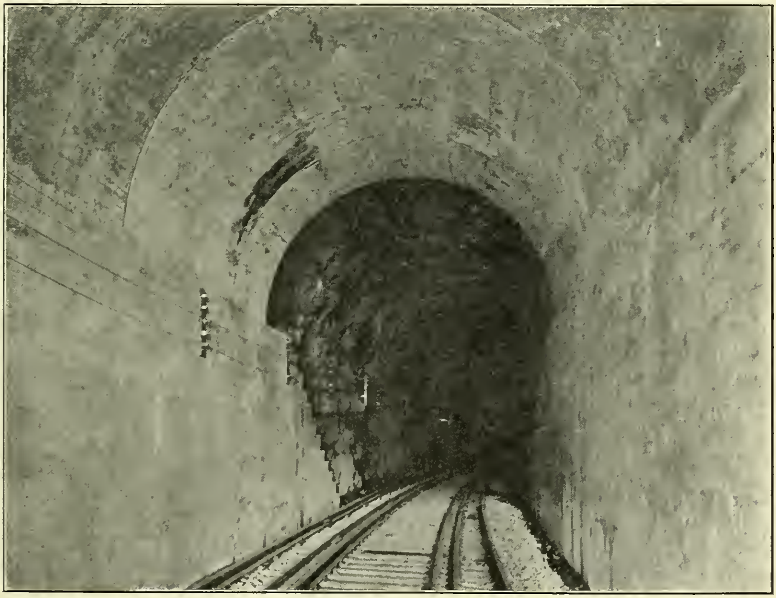


FIG. 2.—SCRANTON TUNNEL; JUNCTION OF PLAIN ROCK SECTION AND CONCRETE-LINED SECTION.

collisions on this piece of single track, and thus trains are passed through with perfect safety.

Costs.—The following are the contract prices for the tunnel proper:

Shaft excavation....................	$7.00	per cu. yd.
Tunnel excavation.................	3.35	" "
Backfilling over timber and behind masonry........................	1.50	" "
Overhaul, 100 ft. in excess of 1 000 ft.	0.01	" "
Class "A" concrete in forms........	9.00	" "
Class "B" concrete in forms........	8.60	" "
Third-class masonry................	6.50	" "
Long-leaf yellow pine..............	45.00	" M., B. M.

The average cost of the tunnel proper for excavation and lining, including the shafts, was $90 per linear foot.

Engineers and Contractors.—Westinghouse, Church, Kerr and Company, of New York, were the engineers for the work, and The Rinehart and Dennis Company, of Washington, D. C., were the contractors.

The execution of the work was under the personal supervision of Mr. P. B. Easterbrooks, Resident Engineer for Westinghouse, Church, Kerr and Company, and Mr. J. H. Rinehart, Second Vice-President and Secretary of The Rinehart and Dennis Company, to whom credit is due for its successful prosecution.

The purchase of the right of way, and the work of track laying, lighting and signaling were conducted by the operating department of the road, under the supervision of Mr. Charles F. Conn, Vice-President and General Manager.

Contractor's Plant and Operations.—Railroad delivery for coal, and a convenient water supply dictated the location of the air-compressor plant at the south end, and the compressor location fixed the situation of the camp, boarding houses and office.

The company's plans required two shafts, and the simultaneous excavation both ways from each shaft, and at the ends, thus making six points of simultaneous attack.

It was calculated that from 24 to 28 drills would be necessary,

and the requisite air supply was approximated at, roughly, 100 ft. per min. for each drill, at 100 lb. per sq. in., and the boiler capacity at 20 h. p. to each drill. There was actually installed one 80-h. p. and three 150-h. p. boilers, all being return-tubular with brick arch, cast-iron fronts and iron stacks, making a total boiler capacity of 530 h. p.

The selection of the compressors was governed by the plant on hand, and comprised one Rand straight-line, 16 by 24 in., of about 600 ft. capacity, two Rand straight-line, 20 by 30-in., each of about 1 000 ft. capacity. At times all these compressors were run at about 20% more than their normal speed of 110 rev. per min., and, roughly, their combined capacity was increased to 3 000 ft. of air per minute when working at their maximum. The three 150-h. p. boilers provided an ample supply of steam, without the assistance of the 80-h. p. boiler, and the results indicate that the preliminary allowance was high, and that about 15-h. p. boiler capacity in this instance was sufficient to compress 100 ft. of air per minute to the gauge pressure of 100 lb.

Contributing to this economical boiler capacity was the use of anthracite coal under forced draft from a steam jet, making practically perfect combustion, and a more than usually tight distributing pipe line.

Whenever a pipe line is to be determined, there is always a balance to be drawn between small size, large friction, and small first cost, and the alternate. Without pretending to any accurate determination of these features, a practical balance for this particular work was to run a main 6-in., sleeve-connected, wrought-iron pipe from the compressor, past the first shaft, and up to the second. This pipe was reduced to 4 in. between the latter and a receiver at the north portal. A 3-in. pipe carried the line to the bottom of each shaft, and the further extension toward the several faces of attack was by 2-in. pipes resting on the bottom. Temporary connection between the end of each pipe and the drills was made by 50 ft. of 2-in. rubber hose, distributed to the several drills by 1-in. rubber hose. The main 6-in. pipe line was laid uncovered in the streets, over and parallel to the tunnel, and was laid during hot weather. On account of its position in the street, the pipe, while slightly sinuous in detail, was very nearly straight in its

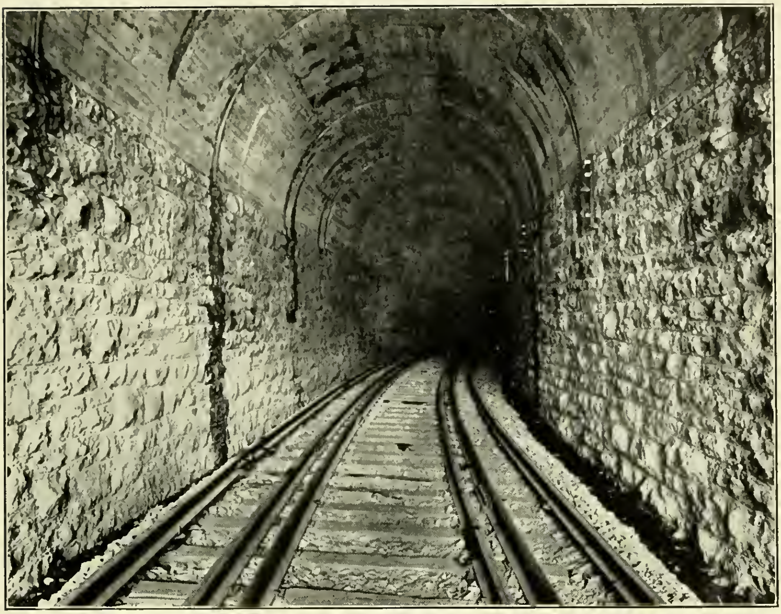


FIG. 1.—SCRANTON TUNNEL; MASONRY-LINED SECTION, WITH RUBBLE SIDE WALLS AND CONCRETE ARCH.

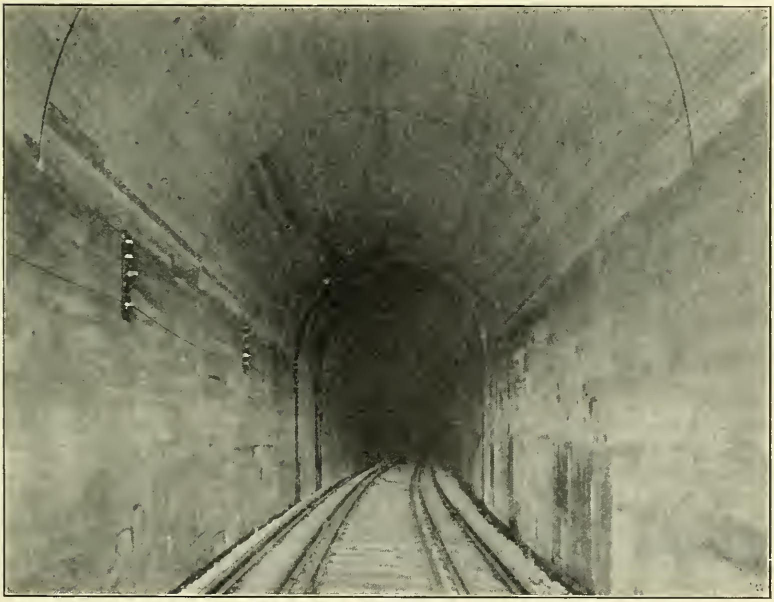


FIG. 2.—SCRANTON TUNNEL; MASONRY-LINED SECTION, CONCRETE THROUGHOUT.

general direction. Expansion bends in the pipe were impracticable on account of its location, and as expansion joints are costly and unreliable, the pipe was laid without any appliance to take up the change of length caused by variation in temperature. The line thus laid gave no trouble in passing through the changes of temperature from summer to winter, and from winter to summer.

The general method of tunneling was to carry the bench and top heading together, with the heading from 50 to 75 ft. ahead. A traveling framework, of half the tunnel width, and with the top at a lower elevation than the top of the bench, was mounted on wheels, which ran on temporary rails. The frame admitted the passage of two cars—one to be run up to the bench at its side, and the other underneath. Either car could be loaded by chutes from the platform, and, at the same time, by shoveling the bench excavation. The connection between the traveling platform and the unexcavated portion of the bench was formed with 4-in. lagging built into a plank 24 in. wide, thus making a wheel-barrow run-way between the face of the bench and the movable platform. The heading spoil was loaded into wheel-barrows, wheeled over the plank and dumped through the chute on the traveling frame, the whole operation being performed without interfering with the loading of the bench spoil.

When it became necessary to blast, the planks were simply loaded on the frame and the latter moved back on its wheels to a safe distance.

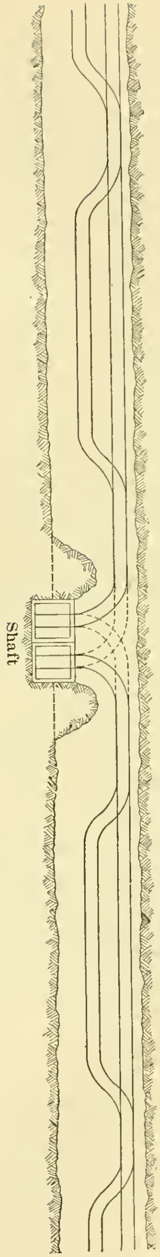


Fig. 3.

CONTRACTORS LAYOUT OF TRACKS AT SHAFTS FOR HANDLING SPOIL.

The foregoing method seems to be so simple and effective as to be scarcely worth description. Other ways of handling bench and heading material at the same time are advocated; nevertheless, the method outlined seems to the writers to be the simplest and cheapest.

A peculiarity of the bench rock, in part of this tunnel, modified considerably, not only the foregoing procedure, but also affected the timbering. That peculiarity was a consequence of the combined hardness and tenacity of the bench material. To break it out required so much explosive that the rock was blown lengthwise of the tunnel with such force as to wreck any permanent timbering erected within 200 ft. In addition, block-holing the spoil was always necessary after the first blasting. The interruption to the work of loading the heading spoil, caused by moving the traveling frame out of danger, was so serious as to render it impracticable to continue the simultaneous excavation of the heading and bench. The heading material, while requiring timber for permanent support, could be left temporarily unsupported; the support being required, not to hold up an overhead mass, but to prevent and support exfoliation, slabbing and weathering of the material. On account of these features, the heading was worked for a reasonable distance ahead, and the force then dropped back and split the bench in two lifts.

The full section of the tunnel was carried without timber support, to the extent of as much as 300 ft. The timbering was then erected from the bottom, and its full section was completed and packed from the floor of the tunnel.

Drilling and Shooting.—The typical plan of drilling the heading was, as shown in Fig. 4, with the eighteen holes fired in the order thereon indicated. From two to six additional holes were often found necessary. The longest holes were the cut holes fired first, which were from 8 to 9 ft. long. The widening holes were from 6 to 8 ft. long. The round was counted to make an advance of from 4 to 6 ft.

With two 3¼-in. machines, the time required to drill the holes was generally 7 hr. The total round drilled averaged about 140 lin. ft. The completion of the loading, wiring, firing by battery in series, reconnecting for the successive blasts, and the delay necessary between them for the explosive fumes to be blown out by com-

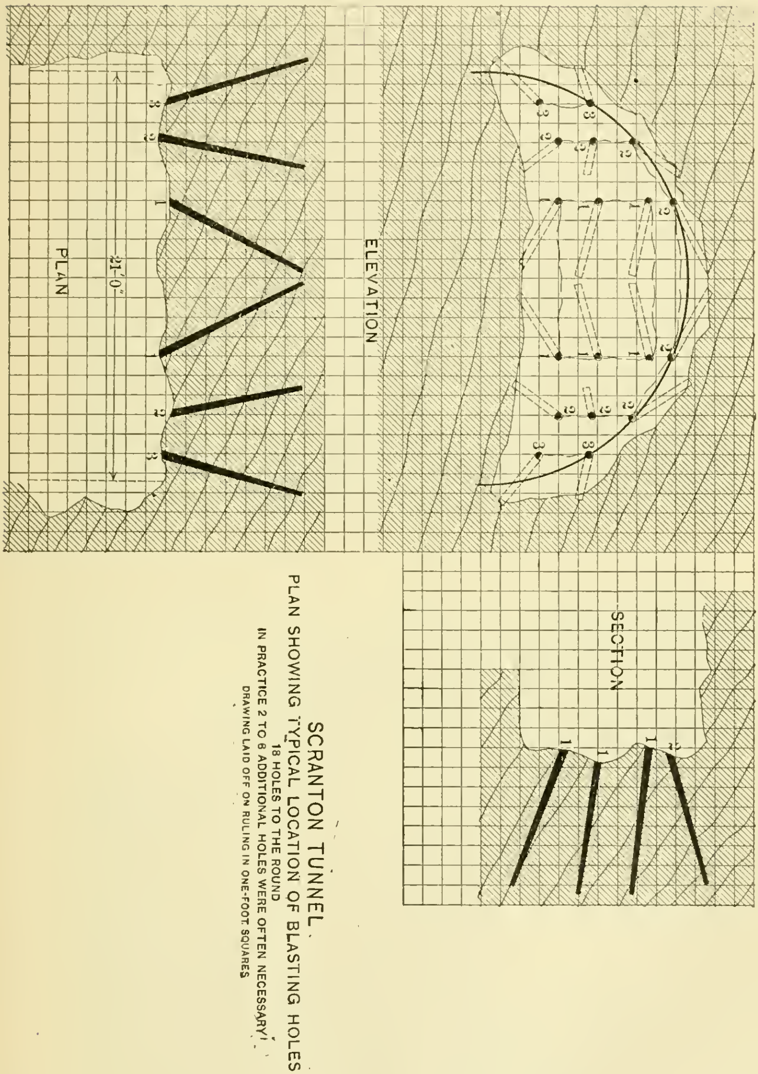

PLAN
21'-0"
ELEVATION
SECTION
SCRANTON TUNNEL.
PLAN SHOWING TYPICAL LOCATION OF BLASTING HOLES
18 HOLES TO THE ROUND
IN PRACTICE 2 TO 8 ADDITIONAL HOLES WERE OFTEN NECESSARY
DRAWING LAID OFF ON RULING IN ONE-FOOT SQUARES

Fig. 4.

pressed air allowed to escape for this purpose, consumed varying times, from 30 min. to 2 hr., or an average of, say, 50 or 60 min.

The delay from shooting was greatest in the portion of the tunnel excavated from the shafts. In this portion the fumes seemed to hang and accumulate, not only on the firing side, but also on the other side of the shaft, producing delays in both places.

The explosive was 40 to 50% dynamite, mainly the latter, and the cost per cubic yard, for explosives, caps, wires, etc., for all the excavation, was equivalent to the cost of 3½ lb. of 40% dynamite.

Cars and Rail.—The type and capacity of the dump cars used in tunnel excavation is a practical question of considerable importance. General conditions require a 3-ft. gauge. The cars have to be handled as single units up to the portals or shafts, and beyond. No method seems to be economical, except by hauling single cars by mules. The car, therefore, should be nearly equivalent to the hauling capacity of a mule, and must be open at the end so that in loading the least lift is required to reach its floor—the dumping will require change or reversal of position. The car, therefore, must be rotary. A car of the following description answers well in practice:

A rigid frame, with four 15-in. wheels, 3½-in. tread, 2½-in. axles; inside bearings, iron rotating spider; bed about 5 ft. 6 in. at back, 5 ft. 8 in. at front, 6 ft. long, with 18-in. sides, all inside measurements. Such a car transports about 1 cu. yd., solid, and weighs, empty, about 2 000 lb. The car is fitted with a removable tail board, like a cart.

A 20-lb. rail, with abundant cross-ties, can be used; but a 30-lb. rail is the more economical, in the end. Rails, in order to be handled down the shaft, should be in lengths of not more than 20 ft. The detail of moving ahead, in order to get the car up to the bench face, is arranged by placing, inside of the fixed rail, loose rails laid on their sides, with their heads against the web of the fixed rail. The car wheel flanges roll on the web of the loose rails, and the latter are slipped ahead at intervals until the excavation permits another length of fixed rail to be laid.

Timbering.—It will be noticed, Plate XX, that there were plans for two methods of timbering. One, the contract plan, was for solid timbering, with provision for concrete arching against its inside perimeter; the other was for voussoir blocks erected inside of

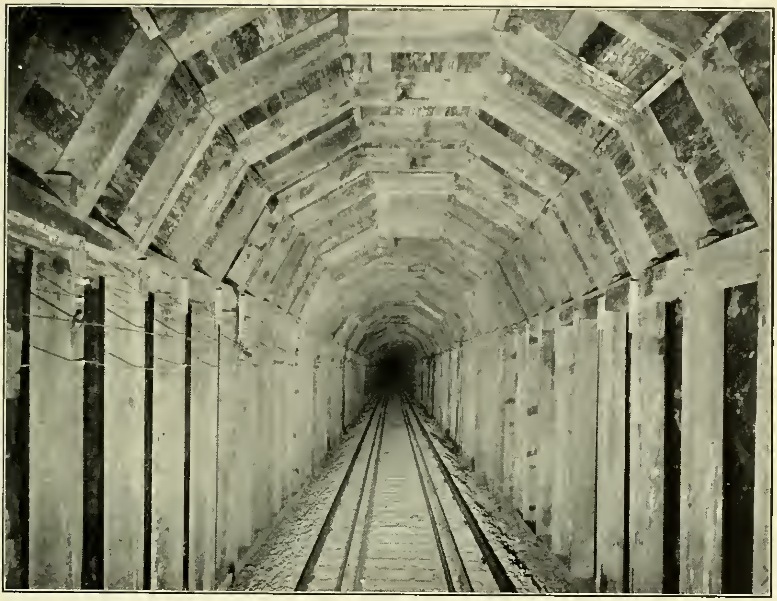


FIG. 1.—SCRANTON TUNNEL; PERMANENT TIMBER-LINED SECTION.

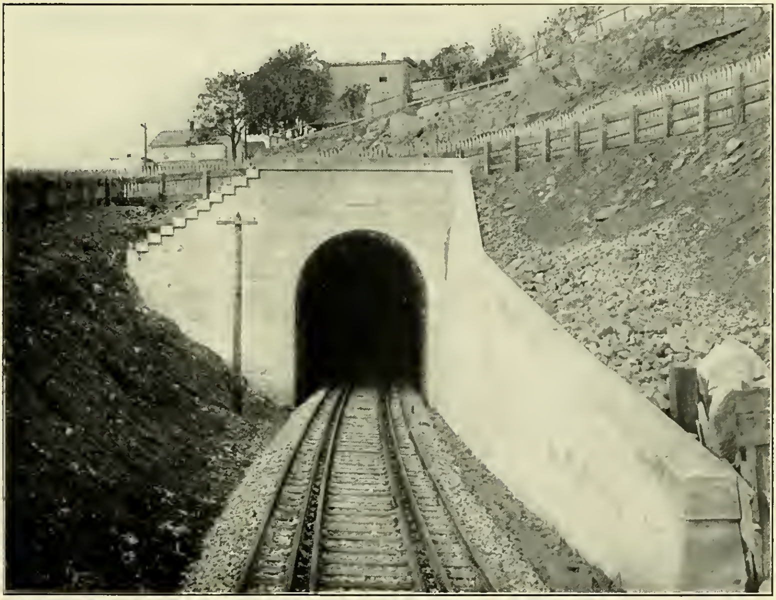


FIG. 2.—SCRANTON TUNNEL; SOUTH PORTAL.

the masonry section and intended to be removed at some future time and replaced by concrete. Timber arching of 10 or 12-in. square arch blocks, with 3 or 4-in. lagging spanning the space between each pair of rims, is the standard of long experience. For economy and ease of erection and packing, and subsequent stability it cannot be improved.

The contract form of timbering, here designated as "segment lagging," for want of a better name, in which the lagging and arch blocks are, so to speak, the same, is ideal in some respects. It uses more timber per foot of tunnel, but gives stronger support per foot, saves in single-track tunnel, roughly, 1 cu. yd. of excavation per foot, and, where the tunnel is to be lined with concrete, furnishes a back form for the concrete, closely concentric to the soffit, thereby saving either an excessive use of concrete to fill the space between the masonry arch and the timber, or avoiding the formation of an extrados for the concrete where the arch is held to a regular thickness. It also avoids the subsequent expense, uncertainty and delay of packing about 1 cu. yd. per lin. ft. between the concrete and the timber.

Pre-supposing a short length of tunnel provided with this timber in place, it would seem to be an easy matter to add to it, with the result of fitting the successive segments to perfect line and stable position. Trouble comes from the variation in the thickness and squareness of commercial timber and from the difficulty of getting a true radial joint. The effect of any inexactness of framing is to carry the bearing on part of the timber and leave other pieces loose. In all timbering the integrity of its form and stability in position when it is packed depends upon the perfect wedging between the voussoir blocks and the perimeter of the excavation, assuming that the wall-plate is immovable in its position. When the arch block and lagged timber are used, the blocks require to be wedged, or propped, to the roof only for every 4 or 5 ft. of the length of the tunnel, then the lagging simply has to be laid on, and the packing space is free to be filled behind and around the props. The operations are few enough to permit the propping to be done thoroughly, without expense and delay.

With the "segment lagging," a very much greater number of pieces require to be wedged up. In the case of the 8 by 8-in. seg-

ment lagging of the plan, as against rims at 5-ft. centers, they are seven and one-half times as numerous per foot of tunnel. This wastes timber in props and wedging. The supports form an almost continuous line, converting the packing space into a number of separate pockets, lengthwise of the tunnel, in which thorough packing is difficult. On the other hand, in any form of timbering, the packing will settle away from the excavation, due to shrinkage of the timber, or shrinkage and readjustment of the packing material from the jar and concussion of blasting, or frequently from the settlement of the wall-plate itself. With the arch-block timbering, there being a free space above the timbering, the tendency of the packing is to settle and slide toward the haunches and open the inside joints of the haunch-blocks. With the segmental lagging, the settlement is confined to the separate longitudinal spaces between the lines of props, and does not become cumulative; therefore, there is less tendency toward deformation.

The experience of the writers has been that the "segment lagging" timbering costs very much more per thousand than the arch-block form. It should also be noted that the yard of excavation saved is not saved at the full price for tunnel excavation. All that is saved is the cost of loading and disposing of the spoil, and of a very small portion of explosive. The cost of all drilling, power and plant, and the general expense and profit, would remain the same per foot of tunnel.

If the company had desired to put in construction or permanent timbering, and to leave sufficient space to put in, at some future time, a masonry lining, without removing the original timber; and if it had had the option of putting in either the "segment lagging" form or the arch-block form of timbering, and could have paid the same unit prices for excavation, timber and packing in either case; there would have resulted a very material saving in cost per foot of tunnel by the use of "segment lagging," and, in the writers' opinion, a very much stronger timber arch would have been obtained during such time as reliance was placed upon it. With the Scranton prices and dimensions, the writers would estimate this saving, for equal masonry clearance, at about $2 per ft., if the same unit prices held.

For the reasons before stated, the contractor's unit prices, if he

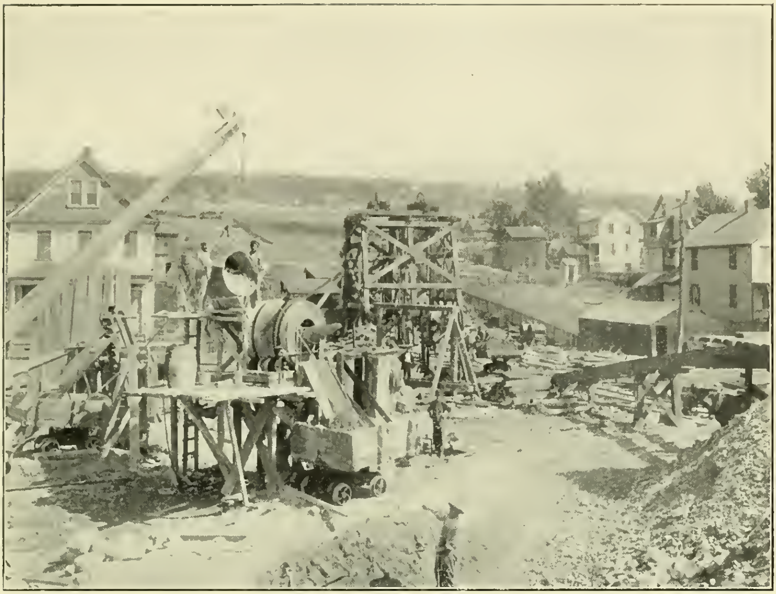


FIG. 1.—SCRANTON TUNNEL; HOISTING RIG, SHAFT NO. 1.

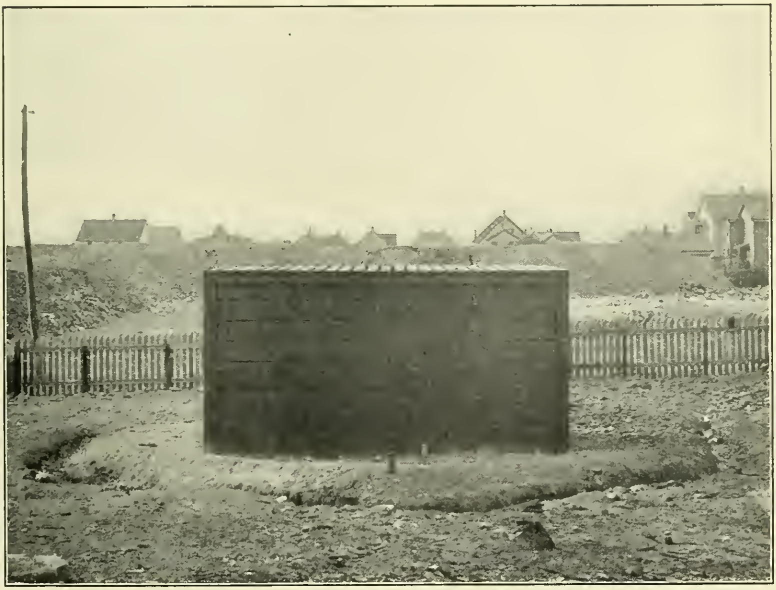


FIG. 2.—SCRANTON TUNNEL; SHAFT-HOUSING.

had the chance to consider the two forms in making proposals, would have to be higher for excavation, timber and packing in the case of "segment lagging." The experience of the writers is that these higher unit prices would more than overcome the apparent saving. In other words, if offered the construction of a single-track tunnel at a given price per linear foot of tunnel, with either form of timbering at the contractor's option, the writers would prefer to use the arch-block form, although it is believed that the cost of the two forms would be nearly equal.

Practical erection difficulties in "segment lagging," in its requirements for exact radial joints and in its multiplication of wedging, would lead to preference for the arch-block form as being more rapid and safe with rougher workmanship.

The foregoing comparison applies only to the method of timbering for average shale or rock tunnel, without consideration of its adaptability to masonry lining to be built afterward.

If the "segment lagging" timber is to be lined afterward with a concrete arch, the practical advantages of this form of timbering, in reference to future concrete, are very great. The advantages are mainly in reference to packing with solid concrete the whole space between the soffit of the concrete arch and the perimeter of the timbering with the minimum quantity of material, and, at the same time, holding very closely to some pre-determined thickness of arch. If the arch-block timber be in place, and the same thickness of arch concrete is required, there is the 12-in. space between the lagging and the extrados of the concrete to be filled in addition. If filled with concrete, there is required an extra quantity of from $\frac{3}{4}$ to 1 cu. yd. per ft.; if with dry packing, the same quantity, and with a support built up of concrete, packing, timber and again packing. By the "segment-lagging" method, there is the original dry packing above the timber, and everything below the timber is solid, making a better job with a saving of the intermediate packing.

Lighting during the Tunnel Excavation.—The writers have generally used gasoline (1-gal. tanks with open burners) for lighting tunnel work on the ground. Its portability, the opportunity of obtaining illumination at the right place, and its general flexibility overcome its high cost.

Gasoline, of course, is dangerous; the renewal of the lamp is a considerable expense, and occasional accidents are bound to occur. The heat, also, is very objectionable.

The accounts show that the cost of lighting, night and day, inside and outside of the tunnel, was more than $6 000, or, roughly, say, 6 cents per cu. yd. By months, it represents $500 per month for double-shift work, or, in another form, about $1.25 per ft. of tunnel.

The contractors returned to the use of gasoline after a previous experience with a tunnel of similar length but smaller section, driven with the use of the electric light. As the tunnels were of greatly different sections, and varied almost totally in material and method, the comparison is not at all fair, but electric light on that work cost about 11 cents per cu. yd. With the electric light there is a perpetual nuisance in replacing wires and broken globes, and it is troublesome to concentrate the lighting where it is needed most—right up in the heading at the front.

It is believed that, on an average, there will be found no material cost advantage in one method over the other, and the advisability of the method to be adopted will have to be settled by its facility in use. The writers have thought, heretofore, that the verdict was in favor of gasoline, but now believe that the electric difficulties can be overcome in part, so that, at least for long tunnels, it will be the better method.

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PAPERS AND DISCUSSIONS.

THE CHANGES AT THE NEW CROTON DAM.

Discussion.*

By Messrs. Alfred Craven, George S. Rice and Charles S. Gowen.

Alfred Craven, M. Am. Soc. C. E. (by letter).—One should not be permitted to infer, by a perusal of the *Transactions* of this Society, that one of the greatest structures of its kind, designed and partially carried to completion by one of its most distinguished members, had been a failure to such an extent as to necessitate its partial demolition and subsequent reconstruction on different lines, without a full discussion of all the reasons that really brought about the change. Such a conclusion as to failure would undoubtedly be arrived at by one familiar only with Mr. Gowen's paper, "The Foundations of the New Croton Dam," presented on February 21st, 1900, on learning subsequently of the radical changes which have since been made. Mr. Craven.

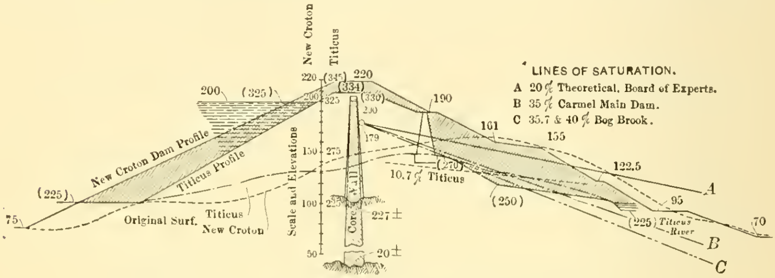
Fortunately, Mr. Gowen has now supplemented his original paper by the discussion of the changes made, giving substantial reason why, in his opinion, they were unnecessary.

William R. Hill, M. Am. Soc. C. E., who was primarily responsible for the radical changes made, has replied to Mr. Gowen's later paper, giving reasons why, from his point of view, the changes were advisable; he has given his views, as heretofore frequently repeated in the technical journals as well as in the daily papers, and supports these reasons by noting their endorsement by a Board of Expert Engineers, and others.

* Continued from February, 1906, *Proceedings*. See December, 1905, *Proceedings* for paper on this subject by Charles S. Gowen, M. Am. Soc. C. E.

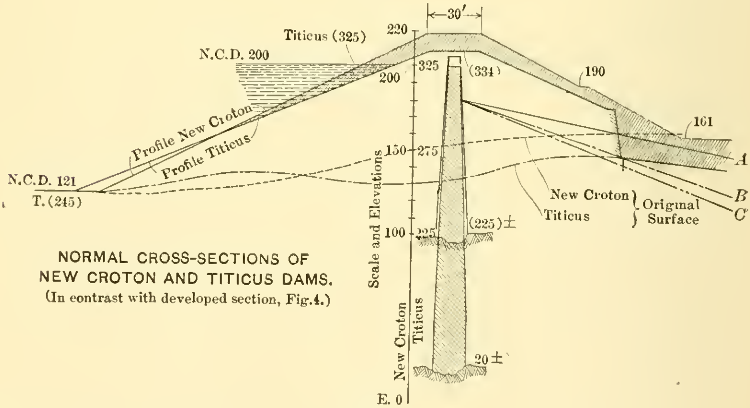
Mr. Craven. It appears to be only proper, therefore, that the views, as to the changes in construction, of the engineer who designed this great work, the late Alphonse Fteley, Past-President, Am. Soc. C. E., should also have their place in this discussion.

Fortunately, Mr. Fteley's views are on record elsewhere. In *Engineering News* of December 12th, 1901, they may be found in full; herein will be noted such quotations, only, as bear particularly



COMPARATIVE PROFILES OF NEW CROTON AND TITICUS DAMS
(These are developed profiles on "lines of least resistance to passage of water." Elevations in parenthe es relate to the Titicus Dam.)

FIG. 4.



NORMAL CROSS-SECTIONS OF NEW CROTON AND TITICUS DAMS.
(In contrast with developed section, Fig. 4.)

FIG. 5.

on the main questions involved. The writer also reviewed the subject at some length in *Engineering News* of January 12th, 1902.

Mr. Hill, in his argument on Mr. Gowen's recent paper, notes that Mr. Gowen has referred to the above-mentioned "rejoinders to the Committee of Experts without giving any information as to the contents of these rejoinders."

To avoid long technical descriptions and repetitions of arguments, and trusting somewhat to the reader's careful perusal of the

papers mentioned, the writer has reproduced here, in Figs. 4 and 5, his own drawings from *Engineering News* of January 12th, 1902. Mr. Craven.

Mr. Fteley, after calling attention to the hardpan formation in which the core-wall was to have been built, says:

"In instances of this kind, when a rock foundation is found within accessible distance under the central part of a dam, it is very usual to abandon the rock foundation and to let the foot of the core-wall step up into the earthy materials of the side hill."

He then cites the embankments of Bog Brook, Carmel Main Dam, Titicus "and many others" (see Fig. 3, page 609, of December *Proceedings*), and states:

"In the present case, however, the core-wall was extended down to the underlying rock into which a trench was excavated to receive the foot of the wall; this arrangement presents the additional advantage of establishing the high wall on an unyielding foundation.

"At the southern end of the central body of masonry where the embankment begins, the height of the surface of the side hill originally stood at an elevation of less than 60 ft. below the higher water mark of the reservoir; from that point south this depth gradually diminishes down to nothing. A large excavation has been made in the side hill to accommodate the earth slopes of the pits necessary for the construction of the masonry dam and of the wing-wall; these pits are obviously to be refilled, and it is through them that the large section, Fig. 2,* of the report is made."

Mr. Fteley, after taking exception to Fig. 2* of the Experts' report, which he very properly says "will convey an idea very different from what the facts will warrant," which indicates the refill of this pit as being a part of the earth dam, and which is furthermore taken so seriously by Mr. Hill, says:

"I may here point to the fact that, when building Titicus Dam, an extensive earth excavation was also made into the side hill on the north side of the masonry section in order to establish its footing and that of the adjacent core-wall on the rock foundation and, if a section of the structure were made on a similar 'line of least resistance' it would show the core-wall with earth embankments on each side, on a minimum slope of $1\frac{3}{4}$ to 1 with a height of 100 ft. A profile made on this 'line of least resistance' at Titicus Dam, in juxtaposition to profile No. 2,* would show a result in favor of the New Croton Dam."

The writer will here call attention to the fact that the Titicus Dam, which will be frequently referred to, has been taken generally as a matter of comparison, as it approaches more nearly to the New Croton Dam in its general features than any other dam of composite type.

* Fig. 2 of the Experts' Report is referred to here.

Mr. Craven. Mr. Fteley then considers the functions of an earth dam with core-wall, referring particularly to the case in point:

"Let us consider the two embankments of the dam separately, as they are called upon to act in a very different manner.

"The up-stream embankment will be in the water, and the (lower and larger part of it will repose, not against the core-wall, but against the main dam) from *A* to *B* (see plan herewith). When fluctuations occur in the reservoir, they will be so slow, on account of its vast area, that it may be said that no water will flow through the bank with sufficient velocity to displace any particles of earth; the only conditions left to be fulfilled are, consequently, that the embankment will be sufficiently water tight, and that it will not slough off or be washed off on the surface; the last condition will be met by covering the slopes as shown in the plans and specifications, with a layer of broken stone with heavy paving upon it.

"As to the condition of water tightness, it is sufficiently met, in my opinion, by the character of the materials to be used, which I have observed continuously for several years while refilling the excavation for the main dam.

"The up-stream bank of Titicus Dam, which is nearly 100 ft. high, with a similar slope, was built with materials finer than those used at Croton Dam, and, barring a slow and perfectly regular vertical settlement, which was expected, has acted in a very successful manner. For several years, whenever the height of the reservoir permitted, exact measurements were taken and the slope has never shown any mark of disturbance."

He then reviews the question of the materials used in the refill and embankment, as follows:

"Before construction, both at Titicus and at Croton Dam, earth excavated from the places where it was expected to take it for refilling, was dumped in the stream, care being taken to form the dumps of the finest materials. In no case did those dumps, left unprotected, show a slope steeper than 1½ to 1. A bank, standing at repose in water, cannot be compared to such hill sides as the experts have observed in the valley where they were acted upon by ground water; if their comparison in that respect were correct, no slope of any kind would be practicable, and not an embankment of the dams built in the Croton Valley and elsewhere, under similar circumstances, would remain standing.

"As to the resistance of such an embankment to the percolation of water, it is obvious that absolute water tightness cannot be expected; and the experiments made at the Cornell Hydraulic Laboratory only illustrate that point; at any rate, it cannot be expected that the results of laboratory tests, on very small volumes of materials collected on the ground, can throw any valuable light on the ultimate behavior of an extensive bank through which the water would have to percolate for a considerable distance before reaching the masonry parts of the structure. Adequate knowledge of the materials used or to be used, and experience, must be depended upon to pass judgment on those matters; moreover, a comparison with

the results obtained in the case of the dams built in other parts of the valley show that sufficient water tightness can be confidently expected from the proposed embankment." Mr. Craven.

The writer will remark here that the Board of Experts, while objecting to the quality of the materials used in this embankment and fill, remark:

"All the tests indicated that this material, which we found to be almost identical in character with that which has been used in the construction of all the earthen dams in the Croton Valley, is permeable to water under any head from 3 to 150 ft., and that when exposed to the direct action of water it disintegrates and assumes a flat slope, the surface of which may be said to be slimy."

This statement (a portion of which is quoted by Mr. Hill), while evidently intended to be condemnatory of the material used, might well be considered, in view of the fact that of the ten earth embankments of the dams in the Croton Valley, all have successfully stood the tests from 10 to 25 years, as fully proving its unquestionable value in an embankment.

Mr. Fteley then reviews the functions and conditions for the down-stream embankment:

"The conditions under which the down-stream embankment would have to perform its functions would be entirely different. Nothing need be said of the water that may enter the bank from springs in the side hill or from the rain; the conditions in that respect will be the same as have always existed, with the difference that the turf on the surface will shed the greater part of the rain. There remains the water which will find its way through the masonry or through supposed deep fissures in the rock formation. As no water is expected to pass through the central body of masonry, the surface to be considered is limited to that part of the core-wall adjacent to the refilling of the excavations or to the embankment. What amount of water can find its way through the wall at that point can be appreciated from a comparison with the other dams mentioned in the report and from the comparative thickness of the masonry. In the majority of the cases referred to, the walls are of less thickness, and although the lower embankment will contain, as must be expected, a certain amount of water, the tests made by the experts indicate that a very small volume of it will flow through the wall. At Titicus Dam, where the wall has more thickness, the indications are that very little, if any, water finds its way through it; in the present case the wall, for the greater part of its height, at the points where the pressure is highest, is 18 ft. in thickness and built of excellent masonry. From these considerations the conclusion is consistently reached that, in view of the character of the up-stream bank and of the core-wall, a very small amount of water will reach the down-stream bank from those sources."

He then takes up the question of danger from saturation of the down-stream embankment, and questions the propriety of the arbi-

Mr. Craven. trary selection by the experts of the conditions in the outer bank of the Middle Branch Reservoir as a guide on which to base an arbitrary line of saturation—to govern in such cases, saying:

"The experts show a certain* 'line of bank saturation' as that likely to obtain in the present case. They base their statement on the observations taken by them at the various dams built in the Croton Valley; they find that the maximum safe height of an earth embankment with slopes of 2 to 1 would be 'on the bases of the loss of head and saturation at Middle Branch, 63 ft.; Bog Brook, 100.6 ft.; Titicus, 82.3 ft.; Amawalk, 72 ft.; Carmel Main Dam, 102.5 ft.' I fail to understand on what basis they state that from their observations the high embankment adjacent to the masonry dam would nearly approach the Middle Branch rate; such a conclusion would presuppose a complete knowledge of the comparative materials used, of the quality of workmanship, and of the various conditions existing during construction, which cannot now be obtained, as the Middle Branch Dam was built more than twenty years ago.

"Titicus Dam, with its high embankment and its heavy core-wall, is the structure to which can be best compared the New Croton Dam in several respects, and the experiments show that very little water, if any, finds its way through the core-wall, the water in the outer embankment standing 40 ft. below the reservoir level. A similar result is expected in the present case, and should a small amount of water find its way through the wall, the lower embankment, which is to be formed of comparatively porous materials, would allow of sufficient drainage, inasmuch as (to quote from the expert's report), 'the slope of the surface of the saturated earth in the bank is determined by the solidity of the embankment.'

"The Auxiliary Carmel and Titicus Dams show very favorable results, although the lower banks were formed of fine materials, none others being found within reach; with comparatively porous materials they would have shown steeper slopes of saturation. I cannot see the truth of the statement that 'the more compact the material of which the bank is built, the steeper will be the slope of saturation.' With compact material, the sectional area of flow is larger below a given level than with porous material, and as the bank slope is one determining factor of the line of saturation, this line tends to approach the slope line. With porous material in a down-stream bank the slope of saturation is steeper and the area of flow less. Unless the water finds its outlet on the face of the slope of the embankment, the slope of saturation will also be regulated by the fact that it will reach the ground-water level at a point near the toe of the slope.

"At the New Croton Dam, the down-stream bank has a 2 to 1 slope for a depth of about 60 ft. below high-water mark, and the unusual width of the top of the embankment is equivalent to a flattening of the slope. Below this, the retaining slope along the down-stream face of the main dam (on the 'line of least resistance') has a general inclination of nearly 3 to 1.

"The experts suggest that the alleged lack of stability would be,

* These lines of bank saturation are shown on Figs. 4 and 5.

to a large extent, overcome by flattening the embankment or by facing the lower slope with a revetment of heavy stone paving; these two suggested additions are, in my opinion, unnecessary; if I were to suggest an improvement to the present plan, I would recommend the drainage of the lower parts of the embankment; this work could be done on an extensive scale, at a comparatively trifling cost, with excellent results." Mr. Craven.

After discussing other features of the masonry dam, Mr. Fteley concludes as follows:

"Economy of design, when properly applied, is one of the main principles of engineering; it was undoubtedly given due weight in this instance, and it should not be departed from without the clearest demonstration that the proposed change is a necessity. It is not thought that the experts' arguments would produce that conviction on those experienced in the construction and maintenance of earth dams, and their determination of a probable line of saturation does not appear to be logically deduced from their observations of existing dams, or to be based on a sufficient study of the mode of percolation of water through fine materials."

Mr. Hill remarks:

"Although this paper is entitled 'Changes at the New Croton Dam,' it treats of only one of the several changes that were made in the plan."

In reference to other changes, Mr. Fteley remarks:

"The experts object to the original plan of the dam, which they call unjudiciously designed, on the grounds that no provision had been made to meet the contingency of a sudden and exceptional flow of water due to a cloud-burst or to other causes. The fact is that the original design was especially devised to meet that condition. Although the surface of the reservoir, covering thousands of acres, is so large that it would have a great equalizing power, the contingency of a sudden flood causing the overtopping of the masonry dam was carefully considered. To that effect, the top of the embankment was kept much above the crest of the masonry and a large amount of rock from the excavations was ordered to be placed on the top of the filling, below the dam. to prevent a harmful disturbance of the surface. The contract drawings are not at hand, but it is well understood that they are of a general character and cannot be expected to show all the details of the work, but the connection of the top of the embankment with the crest of the dam is shown on Sheet 22 of the Report of the Aqueduct Commissioners of Jan. 1, 1897, and the necessary orders for the performance of the work were given.

"I learn by the report of the experts that changes have recently been made, one being the raising of the crest of the masonry dam. The reasons for that change in the plan are unknown to me. I agree that it is injudicious, as it destroys a feature which was considered very important. The calculations for the stability of the dam were made in view of the original elevation of the crest."

Mr. Craven. The experts themselves say:

"The masonry dam should not in any case be built higher than was originally designed. Such a change destroys the harmony and efficiency of the design which, having been scientifically determined, should be rigidly adhered to."

Still, Mr. Hill persisted in raising the masonry dam in spite of this admonition; in other words, the views of the Board were to be given weight only in so far as they agreed with his own.

The writer will now comment on the subject from his own point of view.

Unquestionably, the vital point to consider, and the one which should receive most serious thought, is the integrity of the downstream bank of an earth dam.

If formed of too fine material, the flow of water through the bank, assuming that some will pass the core-wall, will be retarded, and the upper plane of saturation will take a flatter slope than through more porous material, giving a greater area of saturation, and may eventually reach the outer slope of the embankment, causing a sloughing off of the material and thus endangering the structure. If this is guarded against, the bank is in no danger; hence, while the outer bank is secure, the dam is safe, even though there may be considerable settlement.

The experts, discussing the question of saturation, to which they properly gave paramount consideration, established an arbitrary and extreme line of saturation, which indicated that the foregoing condition of liability to sloughing of the bank might result.

They concluded, however, that this assumed condition for the New Croton Dam "might be overcome to some extent by flattening the slope of the bank * * * so as to bring the probable slope of saturation not less than 10 ft. below the surface of the bank."

Mr. Fteley suggested, but did not consider it essential, that the same results could be attained by draining the outer bank. It seems clear, therefore, that either of these simple methods might have been followed, thus removing the slightest cause or necessity for the great expenditure of money and time which have resulted in the methods followed—the cost would probably have been less than $50 000, as against $1 000 000 actually expended—and the loss of time in completion, amounting to two years or more, would have been obviated. Why was this not done? The experts say "it would add largely to the cost and would disfigure the appearance of the dam."

In discussing the question of saturation, Mr. Fteley takes exception to the views of the experts wherein they contend that "the more compact the material of which the bank is made, the steeper will be the slope of the saturation."

The saturated portion of the dam is simply that portion below

the inclined plane of the surface of the water in the bank, whether the material be ever so coarse or ever so fine; porosity is merely a degree of compactness or *vice versa,* and all bank material will absorb water. Mr. Craven.

Slope implies motion in water, and there is no absolute retention of water in the outer bank of a dam having its base below the plane indicated by the loss of head in passing through the inner bank and then through a further obstruction of either masonry or puddle. It is simply a partial retention, with motion through the bank, governed entirely by the degree of porosity of the material, and, unquestionably, the more porous the material in the bank, the steeper will be the slope at which water will pass through it. Just the contrary is claimed by the Board of Experts in their report.

Comparing the earth portion of the New Croton Dam generally with that of the Titicus Dam, referring to Figs. 4 and 5, and accepting the definition of the effective height of a dam embankment as "the difference between the level of the water at high-water mark and the level of the point of intersection of the down-stream slope and the plane of the valley bottom," it is evident that the New Croton embankment is only about 30 ft. higher than that of Titicus Dam, instead of twice as high. It is true that the New Croton core-wall has nearly twice the height of that at Titicus, but in the very nature of this case, where both walls are carried to the underlying rock, this difference in height is more apparent than real; more properly speaking, the difference is in depth below the base of the dams.

It will be seen by Fig. 4 that there is only 30 ft. difference in the embankment height: The New Croton has an interior slope of 2 to 1, Titicus of $1\frac{3}{4}$ to 1; the New Croton, on its outer slope, has a variable profile much increased in value by wide bermes carrying it far beyond the profile of the Titicus, which also has a variable profile abruptly broken by a wing wall, which was not intended as, and is not, a barrier to filtration.

This wall has only a shallow foundation, and, as will be seen by reference to Fig. 4, the slope of saturation for the Titicus Dam, as established by the experts, will pass under this wall and out through the restored surface beyond, which, in accordance with their theory, renders the bank unsafe.

The excess of the New Croton profile over the Titicus, as may be plainly seen, is due to its greater width at the water line; the New Croton having a width of 115 ft., the Titicus of only 72 ft. These measurements are on the delevoped "lines of least resistance." (The actual widths on normal sections are 110 and 74 ft.; see Fig. 5.)

This excess in the New Croton is divided between the outer and inner banks, and it shows largely in favor of the New Croton Dam, as was stated by Mr. Fteley.

Mr. Craven. Fig. 5 shows comparative sections of the two dams at right angles, or normal, to the slopes, taken, in each case, near the dividing lines between the earth and masonry dams. It represents truly the actual differences in sections, which differences, as in the case of the developed section, are unquestionably in favor of the New Croton embankment. It shows a dam which, by the definition of effective height, is only about 40 ft. high. This is just south of the heavy wing wall.

The developed profiles on "lines of least resistance" (Fig. 4) have been used in making comparisons only for the reason that they are a creation of the experts, and it is the only method by which the New Croton embankment could be made to appear higher than any of the dams now in successful service in the Croton Valley.

Assuming, however, that the outer refill is to be treated as a part of the bank proper, then, on Fig. 4, are produced the several slope lines of saturation as determined by the experts for the other dams.

The theoretical 20% line of the experts would indicate a danger point on the developed profiles, while the lines of Bog Brook and Carmel Dams indicate absolute safety, passing far below the 10-ft. limit of the slope of the bank, a limit of safety fixed by the experts, which in itself is rather an excessive requirement.

On the other hand, all the lines indicate unsafety in the Titicus profile. These lines are plotted with the experts' assumption of a loss of head of 17% of the depth of water in the reservoirs.

The writer agrees fully with Mr. Fteley that there is nothing whatever to justify the adoption, by the experts for New Croton Dam, of the Middle Branch line of saturation as against the lines of Bog Brook and Carmel.

The experts admit that the core-walls and embankments of Amawalk and Middle Branch were not as carefully constructed as those of the other dams, yet they arbitrarily select the Middle Branch line of saturation to apply to the New Croton Dam.

For the purpose of showing his alleged "inadequacy of embankment" of the New Croton Dam, Mr. Hill compares the width of base and slopes with the broad base and flat slopes of the Amawalk Dam. It must be borne in mind that the Amawalk Dam embankment was made in the form of a great earth fill, no attempt being made to compact the material by rolling or ramming, but trusting to the great mass of material to supply the equivalent of more careful methods of construction. During the long period of construction the material was allowed to settle as best it would; therefore, it is not to be taken properly as a comparative construction with the dams built by the Aqueduct Commission where every precaution was taken that is essential in the construction of embankments,

fully justifying the comparatively steeper slopes and more contracted bottom widths. Mr. Craven.

There is one point in Mr. Hill's argument to which he apparently attaches great weight; it is the alleged "unstable foundation of the embankment."

In articles which he has caused to be published elsewhere, he dwells as follows on this feature of the work:

"This embankment was hazardous because of the unstable nature of its foundation. It was founded over a great refilled pit (giving dimensions of pit). It would be impossible," he says, "to refill this pit as compactly as the original ground, hence the safety of the reservoir was dependent, not only on an embankment of a problematic section, but this problematic section rested upon an unstable foundation."

* * * * * * * *

"The water would be afforded freer access through the refilled material of the great pit than it would have in ordinary cases where the wall below the original surface of the ground is in a narrow trench and protected by the original soil."

* * * * * * * *

Also, "a fourth objection, * * * the permeable and light character of the earth of which the embankment was made, but even with the best material, an embankment so constructed would be insecure."

In other words, his contention is that a made embankment cannot be as solid as, and will permit the passage of water more readily than, earth in a natural state.

In his argument on Mr. Gowen's paper, he has repeated these statements in a somewhat modified form.

The well-known facts are, unquestionably, just the reverse of the above. A properly made artificial embankment, either puddled or rolled in layers, undoubtedly contains a greater quantity of material per unit of volume than an equivalent volume of the same earth in its original position, and is less pervious to water. This rule frequently leads to the removal of a considerable quantity of material, and to a refill, often with the same material, rather than build on the natural surface; and the writer, from his knowledge of the conditions at the New Croton Dam, cannot conceive why there should be in that case any exception to this generally accepted rule.

The fact must not be lost sight of that the material replaced below the lines of the natural surface of the ground is simply a refill confined in a great pit, and no amount of reasoning over this condition can make it an embankment.

This refill could be made, and in fact much of it had been made, to a height of about 100 ft. above the low point of the core-wall,

Mr. Craven. with such care as to preclude the possibility of bringing any undue stress on the core-wall that would tend to its rupture, and, furthermore, when it was again finally removed, the wall was found intact.

It is interesting, therefore, to know (see both Figs. 4 and 5), that above this line of refill, while the dam is no higher than that at Titicus, the embankment, as pointed out by Mr. Fteley, is far more in excess, in its factor of safety, than that of Titicus; and the writer does not believe any one will question the adequacy of the latter.

Properly made embankments will not settle materially; in fact, several cases of dams of considerable height can be cited where accurate levels have been taken on the tops of the dams when completed, and again, long after the reservoirs had been filled with water, additional observations have shown absolutely no settlement of the earth banks. In cases, however, where settlement does occur, if due to the action of water, it must mean the displacement of so much water and a consequently greater compacting of the bank material.

In regard to percolation and filtration through sands and soils, it might be well to study the paper on the Bohio Dam, Panama Canal, by the late George S. Morison, Past-President, Am. Soc. C. E., presented before the Society on March 5th, 1902, and the discussion thereon by Frederic P. Stearns, President, Am. Soc. C. E., on the North Dike of the Wachusett Reservoir; they are instructive and also may be considered as pertinent to the subject under consideration.

The late E. Sherman Gould, M. Am. Soc. C. E., an engineer of extended experience in the construction of dams and reservoirs, has written:

"An earth embankment provided with a heavy masonry center wall carried down to a firm substratum, or, failing in that possibility, to a considerable depth below the surface, the depth being in inverse ratio to the compactness of the material, and well bonded into the sides of the valley, forms one of the best and safest dams which can be built."

It has further been remarked that:

"Of the materials used in the construction of dams, earth is physically the least destructible of any. The other materials are all subject to more or less disintegration or changes in one form or another, and in earth they reach their ultimate and most lasting form."

One more reason has been given, by Mr. Hill and the Board of Experts, why an earth dam should not be built more than 100 ft. in height, and that is a lack of precedent. If engineers wait for precedent surely no advance will be made in any form of construction. There is absolutely no reason why they should stop at 70, 100

or 200 ft. in height for an earth dam; it is merely a question of necessity or expediency. The profession of engineering and architecture would long ago have been at a standstill if "precedent" had been waited on. In this case, however, there is no lack of precedent; Mr. Gowen has cited several cases of earth dams, 100 ft. or more in height, which in the opinion of the writer are, all things considered, much bolder in their conception than the New Croton Dam. Mr. Craven.

Again, if "precedent" is to be sought to solve the question, it should be used negatively; if dams of more than 100 ft. in height had been properly built and had failed, a precedent might then have been established as against further efforts.

Had the test wells been made in the Middle Branch embankment 15 years ago; had the same results been obtained as now, followed by the same conclusions; had these conclusions been accepted, then it is self-evident that the Carmel, Bog Brook and Titicus Dams would never have been built, at least on their present lines; still they stand to-day as masterful examples of good construction and contradictions to the theories and conclusions of the experts, and it is so with many other earth dams with no flatter slopes than are proposed for the New Croton Dam.

In conclusion, the writer desires to say that, in his opinion, there was absolutely nothing to fear from the completion of the New Croton Dam on the lines as laid down in the original plans, knowing full well the care with which these plans were developed and were being carried out.

George S. Rice, M. Am. Soc. C. E. (by letter).—The writer is pleased to note Mr. Gowen's careful consideration of the engineering questions involved in the changes lately made in the New Croton Dam. Mr. Rice.

The subject is one in which engineers, especially those employed by the Aqueduct Commissioners since the original investigations of the various sites, have been very much interested. Knowing the extreme care with which this and other sites were investigated, and having a knowledge of the conditions which existed there, as well as the immense amount of work done by Mr. Fteley in connection with the theory and design of high masonry dams, it is a source of regret to note the criticisms of, and the changes in, the plans originally contemplated for this great work. Mr. Fteley's experience in this class of work was probably greater than that of any engineer of the present time, he having been equipped, not only theoretically, but practically, with a knowledge of the subject.

The writer realizes that it is perfectly natural for the engineer, in considering the question of a dam of this nature, to assume instinctively that a masonry dam from one side of the valley to the other would be an ideal solution of the problem; but when one is

Mr. Rice. familiar with the results of the construction of dams in America, more particularly in connection with such large works as the dams for the additional water supply of the City of Boston, and also the extensions in recent years for the water supply in the Croton Valley, the consummate skill with which this work was originally designed is perfectly apparent. This plan contemplated a masonry dam where the foundations were adapted for it, and when peculiar conditions were found at the southerly end of the Cornell site the subject was treated purely with an idea of meeting the conditions thus found.

In the design of this dam, Mr. Fteley's whole idea was to erect a masonry structure, where it was found better in an engineering way, but when, in his judgment, the conditions warranted, he used a core-wall with an earth dam, and showed in this the very best engineering, that is, good construction and economy. His judgment in this matter was based upon the nature of the material in the Croton Valley, with which he was fully acquainted, as he had designed and constructed several dams there. In the writer's association with Mr. Fteley, it was natural that he should have talked with him on this subject at various times, and consequently he was familiar with the reasons and purposes of his method in designing the dam.

In the last few years of Mr. Fteley's life it was a gratification to him to feel that the changes in the dam reflected in no way upon his judgment; he realized that these changes were made by those who did not have a full knowledge of the subject, and under the circumstances could not have had the experience, and that, therefore, the criticisms were illogical.

In the construction of this work Mr. Fteley had the advantage of having assistants who were experts in this particular line; and if Mr. Hill, as Chief Engineer of the Aqueduct Commission, had consulted members of his own staff and followed their advice, he would have had the command of exceptional talent in this class of work, would have profited by such experience, and, in order to carry out his plans, would not have been obliged to go outside for engineering advice. In reference to certain questions relating to the aqueduct work, it has been noticeable that Mr. Hill followed the advice of his consulting engineers only when it seemed to suit his own ideas.

Mr. Hill, in his report to the Aqueduct Commissioners, in speaking of the changes, said:

"I make this recommendation after carefully studying the situation and plan, and I know that I am absolutely right," etc. etc.

Such an assumption, on the part of an engineer occupying Mr. Hill's position, would seem to show that he lacked a full grasp of this subject.

The writer agrees with Mr. Gowen that the changes which have been made in the completion of the dam were made at an extra cost to the city and a delay which would not have obtained if the original plan had been carried out, and, in his opinion, the dam as reconstructed is no better in carrying out its purpose than the earth section as planned in the original design. Mr. Rice.

CHARLES S. GOWEN, M. AM. SOC. C. E. (by letter).—In reviewing Mr. Hill's communication the writer finds it necessary to call attention to the fact that in the paper under discussion it is definitely stated that its particular purpose is to refute the statement made that the foundation of the core-wall was unsafe; accordingly, the special object of the various sections and views submitted, together with most of the text, is to illustrate this point. The general question of the changes advocated by Mr. Hill was assumed by the writer to have been so thoroughly discussed in the engineering press that the reports and articles published are merely alluded to as justifying him in his contention that the changes were unnecessary. The above explanation seems to be called for, as Mr. Hill states that the writer makes two general contentions, the first of which is that the former plan of the dam (that of September 16th, 1896) was adequate, and he then proceeds to discuss and criticise, in this connection, the plans, sections and statements offered in reference to the second and main contention as to the adequacy of the core-wall foundations. Mr. Gowen.

Mr. Hill states that the only cross-section submitted is one of the embankment and core-wall at a point about 170 ft. from the end of the stone dam. Seemingly, he does not bear in mind that this section is to show the conditions at the most questionable point of the dike of granular limestone, concerning which the criticism of the safety of the core-wall foundation was made. He also forgets, apparently, in criticising the developed section, that this was made to illustrate conditions with reference to the same point in the limestone dike, and that it was made on the lines of least resistance (so-called) to percolation, in exact accordance with the plan pursued by the Board of Experts* in their report on the general question of the effective resistance of embankments and core-walls to percolation, and the resulting condemnation of the core-wall plan of the dam. He also says that it is a mistake to state that the up-stream slope of this embankment of the developed section is about 4 to 1. That there is no mistake about this, is apparent when it is noted that in this section the width of the embankment at the top is nearly 200 ft., most of it lying on the upper side of the core-wall line, which, with the slope below, as shown, is the equivalent of more than the slope claimed, particularly when the material lying above any ordinary slope line demanded at this point is taken into account.

* See Report of Board of Experts, *Engineering News*, November 25th, 1901.

Mr. Gowen. Mr. Hill's citation of the Mill River Dam failure, in order to controvert the writer's statement regarding the causes of failure of earthen dams, calls for comment, and in reply it may be said that the report of the Committee of the American Society of Civil Engineers which was appointed to investigate this matter (Messrs. Francis, Worthen and Ellis) states, in view of the crude methods used in the design and execution of this work "it is obvious that this cannot be called an engineering work;" and also, "no engineer, or person calling himself such, can be held responsible for either its design or execution."

In defining his position in the premises, regarding the general question of the changes made at the New Croton Dam, Mr. Hill states that it is not unreasonable to expect that conflicting opinions will arise as to the efficiency of a plan of a reservoir bank. Further on, he states that his recommendation to remove the core-wall was based on his opinion that the plan was inadequate, and he then, further, says in his report to the Aqueduct Commissioners recommending these changes, "I know I am absolutely right." Continuing, he instances the unanimous opinion of the Board of Experts supporting him, and the further concurrence of engineers in the employ of the city who were asked to give their opinions by the Mayor; and he accordingly concludes that, as far as the general public is concerned, great weight has been added to his conclusions.

In view of the foregoing, which, in the main, is a contention of absolute right by Mr. Hill, as to his position in the important matter of these changes, it is certainly proper to call his attention to the fact that this dam was originally designed after careful research and study, and its construction was supervised by an eminent engineer who had a long experience in the construction of high dams of various designs, and whose success in results has not been surpassed. A comparison of Mr. Fteley's professional record, in this respect, with that of any of the engineers whose dicta or opinions have been cited as bearing on these changes, would certainly warrant the claim that there is at least room for argument in this matter. A record of many years of notably successful professional work along certain special lines cannot be ignored, nor can there be assumption of absolute right in conclusions without at least prompting the query as to what qualification or authority there may be to warrant it.

As to Mr. Hill's contention that the first important change in the plan of the dam was made in September, 1896, before his accession to the position of Chief Engineer, and concerning his remarks in relation thereto, attention is called to this point, that this change was provided for in the specifications of the contract, and that the general plan defining the length of the main dam was tentative only. This change may be considered simply as a determination, of the

Chief Engineer who originally designed the dam, of the position of the end of the masonry structure. This determination was reached after the information regarding the foundation material which the excavation afforded had been considered, and the principal motive for making it was the reduction in the height of the embankment above the restored or refilled surface. This height, accordingly, was reduced to about 50 or 60 ft. The question of the depth of the core-wall below the refilled elevation did not enter into the settlement of this question, except as it was incidental to the reduction of the embankment height. It is perhaps well to say that this determination or change involved no question of taking down work already done, and consequent delay, and that all plans for the progress of the work on the dam from the beginning were made in view of the settlement of this question at the proper time. Mr. Gowen.

The extreme height of the core-wall at the point of junction with the main dam was 183 ft., including the depth excavated in the rock for its foundation, which was perhaps 15 ft. At a point 25 ft. south of its junction with the main dam the core-wall height was 166 ft. This is cited to illustrate the rate at which the extreme core-wall height decreased as the foundation rock slope rose to the south.

In considering the general arguments advanced by Mr. Hill in support of this change, it may be said that his description of the conditions planned at the junction of the main dam and the core-wall is practically correct. He neglects to state, however, that the embankment was to be carried to Elevation 220, thus giving it a thickness or width of more than 100 ft. at ordinary high-water mark, and, as above noted, his extreme height of core-wall is too great. The statement that, at the end of the stone dam, its width was 164 ft., is also excessive, 130 ft. being nearer the mark.

In his discussion of the embankment, he claims it to be 150 ft. high. Granting, for the sake of argument, that this refill should all be considered as embankment, if the Board of Experts' definition of its effective height be taken as being determined by the difference between high-water level on the up-stream side and the toe of the bank on the down-stream side, the height of bank due to ordinary high-water level would be 130 ft., and to extreme high-water level it would be 136 ft., this being on the line of the developed section. As to its thickness at the base, conceding, again, for the sake of argument, the refill to be bank, we must also follow the section of least resistance to percolation shown by the report of the Board of Experts, and we have for the embankment a thickness of at least 800 ft., in place of 650 ft., as claimed by Mr. Hill.*

A height of 136 ft. is not excessive, nor is 800 ft. an exceptionally narrow embankment base, even in comparison with existing struc-

Engineering News, December 12th, 1901. Plan in Mr. Fteley's report.

Mr. Gowen. tures. This is particularly evident when the comparison is noted of existing conditions between this proposed plan and the similar conditions at the Titicus Dam, shown in Mr. Craven's article in *Engineering News* of January 6th, 1902. Here it is shown that the effective height of the Titicus embankment is about 25 ft. less than that proposed for the New Croton Dam, and that the increased thickness and dimensions of the New Croton Dam bank more than compensate for its excess in height. This, therefore, in the light of experience, cannot be claimed to be a problematic section, nor does there seem to be any proper question as to the stability of the fill described by Mr. Hill for the pit beneath this bank. This pit is a hole in the ground, the walls of which would surely retain the refilling, which would have been compacted by the tremendous weight of the refill and bank above Elevation 70, and to a degree at least equal to its original density; this, especially when it is considered that the compact hardpan taken from this part of the excavation did not extend to the bottom of the pit, but was underlaid by a thick bed of sand and gravel.

Mr. Hill lays stress upon the experimental character of the core-wall of 200 ft. height, having "no lateral protection or support whatsoever from the original ground." It is assumed that he refers to such support as would be afforded by the sides of a vertical trench sunk in the ground; but it is difficult to see what advantages there could be in the lateral support afforded to a wall in a trench refilled to a depth of 3 ft. or more on either side of the wall, compared with that due to a heavy refill or massive made bank carried up at equal heights on either side and compacted at least in its lower stretches by the great weight of the bank above. A refill, under such conditions, would certainly afford as much lateral support to any wall, however deep or high, as could possibly be afforded by the refill placed by hand in a narrow space between the face of the wall and the side of the trench, however carefully it was done.

The writer, therefore, cannot see that the depth of a core-wall below the elevation which defines the lower limit of the effective height of an embankment has any bearing upon the essential questions involved in this case, if such wall has a proper base.

As to the actual height of the embankment involved in this discussion, the writer cannot concede that that portion of the refill placed on both sides of the masonry dam at the south end below the original surface line has to be considered. The actual height of the embankment to be considered, that portion above the original surface line, was not more than 60 ft., and the refill below, forming, according to the claims of the Board of Experts and Mr. Hill, the remaining 76 ft. of the bank, should be regarded simply as a restoration of the general surface or topography. Experience with com-

bination dams varying in effective height from 25 ft. to the height of the Titicus Dam, which so closely approximates at this point the New Croton Dam, justifies this conclusion. Mr. Gowen.

For the same reason, *viz.*, experience with varying heights of combination dams, the fear that flow may take place along the face of a dam wall beneath the embankment is not warranted.

Finally, Mr. Hill refers to the permeable and light character of the materials used for the embankment, as stated by the Board of Experts, who are quoted as stating that it was permeable under any head for 3 to 150 ft., etc. The writer feels free to say that he cannot understand the meaning of such a statement which might possibly be explained by the unquoted context. However, he has this to say of the embankment material; that it was composed of gravel, sand, sandy loam and clayey sand, properly mixed and compacted, and that it withstood such proper tests as the Board of Experts made, not excepting tests for permeability and slope action under water.

As to the assumption that Mr. Hill's course in changing the core-wall section of the dam was in keeping with the report of the Board of Engineers (Messrs. Shunk, Davis and Croes) who, in 1880, recommended an all-masonry structure for a dam at the Quaker Bridge site, it should be said that there is no proper parallelism to be cited, as the prevailing conditions at the sites of the two structures in question were different.

Mr. Stearns, in sustaining the writer in his contention as to the safety of the core-wall foundation, has referred to the investigations of the Board of Experts regarding the saturation of dam embankments, and to its conclusions that, in a majority of the cases investigated, the indications were that the core-wall offered no greater resistance to percolation than the embankment. If these conclusions are right, it would seem to be evident that the embankments approach the core-walls in density, and that, furthermore, there can be little or no pressure to act at the base of the wall and thus aid percolation under it. Such indications are certainly not prejudicial to the earthen dam as a safe structure.

Mr. Craven's discussion embodies the principal features of his own and Mr. Fteley's papers concerning the changes in the dams, which were published in the engineering press and referred to in the writer's paper. The writer fully concurs with Mr. Craven as to the desirability, under the circumstances, of bringing these matters into the discussion, so long as the general questions regarding the changes were renewed by Mr. Hill.

The comments of Mr. Rice, referring to his professional association with Mr. Fteley at the time the studies for the New Croton Dam were being made, and to his knowledge of the conditions under

Mr. Gowen. which the necessary investigations were carried out, are certainly of value, as they give additional weight to the general belief that Mr. Fteley's professional conclusions were based upon intimate and scientific knowledge of essential facts as well as upon broad views, and that they were accordingly reliable and abiding.

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PAPERS AND DISCUSSIONS.

TEST OF A THREE-STAGE, DIRECT-CONNECTED CENTRIFUGAL PUMPING UNIT.

Discussion.*

By Messrs. W. B. Gregory, H. F. Dunham and Philip E. Harroun.

W. B. Gregory, Esq.† (by letter).—The writer has not had any experience with three-stage pumps, but, during the summer of 1905, he tested a two-stage, direct-connected, centrifugal pumping unit. This pump had 6-in. suction and 4-in. discharge pipes, and was driven by a direct-current motor on the same bed-plate. The discharge head was measured with a pressure gauge on the discharge pipe near the pump. The suction was measured with a vacuum gauge on the suction pipe near the pump; both gauges being carefully calibrated. Mr. Gregory.

The total head was ascertained by reducing these observations to feet of water, correcting for the difference of level, and adding to the difference between the velocity heads in the discharge and suction pipes. Thus the pump is given credit for the velocity head it produces, as well as for pumping the water against pressures equivalent to the given heads.

The water discharged from the pump was measured by an 18-in. Cippoletti weir, placed in a tank with baffle plates arranged so that the water flowed quietly to the weir. The depth of the water over the crest of the weir was measured by an accurate hook-gauge.

The electrical readings were obtained from carefully calibrated instruments. The electrical losses were measured and corrections made.

* Continued from February, 1906, *Proceedings*. See December, 1905, *Proceedings* for paper on this subject by Philip E. Harroun, M. Am. Soc. C. E.

† Irrigation Engineer, Office of Experiment Stations.

Mr. Gregory. The friction in the pump and motor was obtained when the pump was not primed, and in getting the efficiency of the pump one-half was charged to each, assuming the friction to be constant for all loads.

TABLE 3.—TWO-STAGE TURBINE PUMP.

Time.	Revolutions per minute.	Volts.	Amperes.	Electrical horse-power.	Suction, in feet of water.	Discharge, in feet of water.	Total head, in feet of water.	Depth over weir, in feet.	Discharge, in cubic feet per second.	Water horse-power.	Efficiency of pump and motor; percentage.	Efficiency of pump; percentage.	Efficiency of motor; percentage.
9:50...	1 000	140	101	18.95	2.20	136.2	142.8	0	0	0	0	0	0
9:55...	978	138	126	23.33	3.06	140.0	147.6	0.241	0.597	9.98	42.8	50.2	85.3
10:00...	967	136	176	32.11	6.46	127.3	139.1	0.322	0.923	14.52	45.2	51.9	87.1
10:03...	955	134.3	194	34.95	8.72	115.7	130.3	0.362	1.101	16.25	46.5	53.1	87.6
10:06:..	960	134	208	37.38	10.20	104.2	120.7	0.378	1.216	16.62	44.4	50.6	87.7
10:09...	960	134	217.3	39.05	12.23	92.5	111.5	0.408	1.316	16.60	42.5	48.3	88.0
10:12...	956	133.5	225	40.26	13.48	81.0	101.6	0 427	1.409	16.20	40.2	45.7	88.1
10:15...	956	133.5	236	42.25	14.95	69.4	92.0	0.446	1.507	15.67	37.1	42.0	88.3
10:18...	956	133.5	242	43.32	16.20	57.8	82.0	0.459	1.571	14.56	33.6	38.1	88.3
10:21...	949	132.5	247.5	43.97	18.01	46.3	72.7	0.476	1.657	13.62	31.0	35.1	88.3
10:24...	949	132.5	254.2	45.17	19.37	34.7	62.9	0.489	1.727	12.30	27.2	30.8	88.4
10:27...	949	132.5	260.5	46.31	21.01	23.1	53.1	0.495	1.759	10.57	22.8	25.8	88.4
10:30...	949	132.3	264.0	46.88	21.53	16.2	46.7	0.495	1.759	9.30	19.8	22.4	88.4

Observations were taken, beginning with the discharge valve closed, then opening it slightly, and again taking observations as soon as all the conditions were constant. This process was continued until the discharge valve was wide open.

Some trouble was experienced with the thrust-bearing on account of heating, but the test was not interfered with seriously.

The discussion of the efficiencies of centrifugal pumps in general is of great interest. The writer has had considerable experience in testing a great many varieties of these pumps during the last ten years. Before entering into a discussion of results, it will be well to define exactly what is meant by the term efficiency, which, in general, is the ratio of the output of a pump to the energy furnished to the shaft. With pumps used for elevating water it seems to be fair to use, for the output of the pump, the useful work, or a certain number of pounds of water actually raised through a distance, measured in feet. If the first quantity is expressed in pounds per second, the product of pounds by feet of head, divided by 550 will give the horse-power corresponding to the useful work of the pumps. Usually, the water is discharged with appreciable velocity, and, if the total output of the pump, including the piping, is to be

Mr. Gregory.

credited to the pump, the head equivalent to the velocity of discharge must be added to the height through which the water is elevated to obtain the total head. By either of the foregoing methods, the loss at the entrance to the suction pipe, and the friction losses in the suction and discharge pipes are charged to the pump.

In testing pumps in which the velocities are high, as in the case of hydraulic dredges, and in any case where the pump is to be eliminated from its special arrangement of piping, the total energy developed by the pump is desired, so that comparisons may be made of the pump performance only. In such cases the pressures are obtained near the pump on the suction and discharge pipes, and the

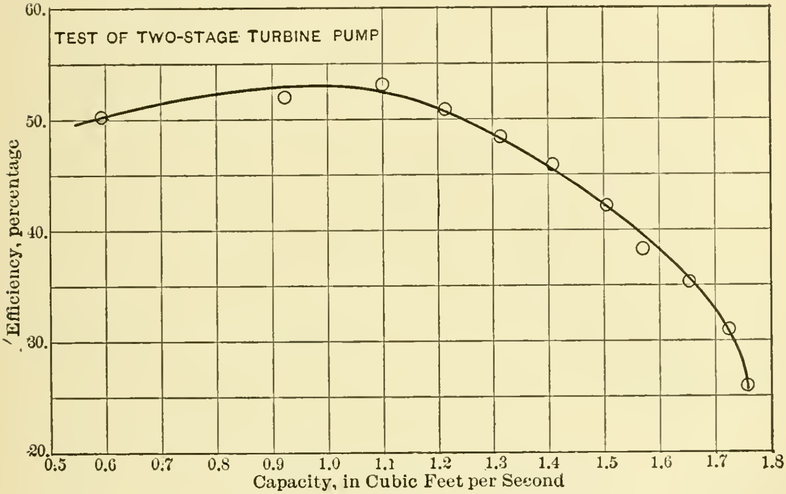

FIG. 4.

quantity of water discharged is measured. From these quantities the velocities in the suction and discharge pipes become known, and the total output of energy may be computed in the manner described in the test of the two-stage pump, as stated previously. When low velocities are used, the difference between the total energy and the useful work will often be small, particularly if the suction pipe is enlarged where the water enters it and the discharge pipe is enlarged where the water leaves that pipe. With the high velocities used in dredging, the difference between the total energy and the useful work is much greater; often the discharge level is practically that of the suction side, and the useful work consists in developing a high velocity and consequently great kinetic energy in order that material may be carried along with the water. It is evident, there-

Mr. Gregory. fore, that, in the case of dredge pumps, the total-energy basis must be used.

It is customary to report tests of ordinary pumping plants on the basis of useful work, and tests of dredge pumps on the basis of the total-energy output. This should be kept in mind in comparing efficiencies.

The writer has tested large pumps, used for elevating water, which had efficiencies as high as 75% when based on useful work.

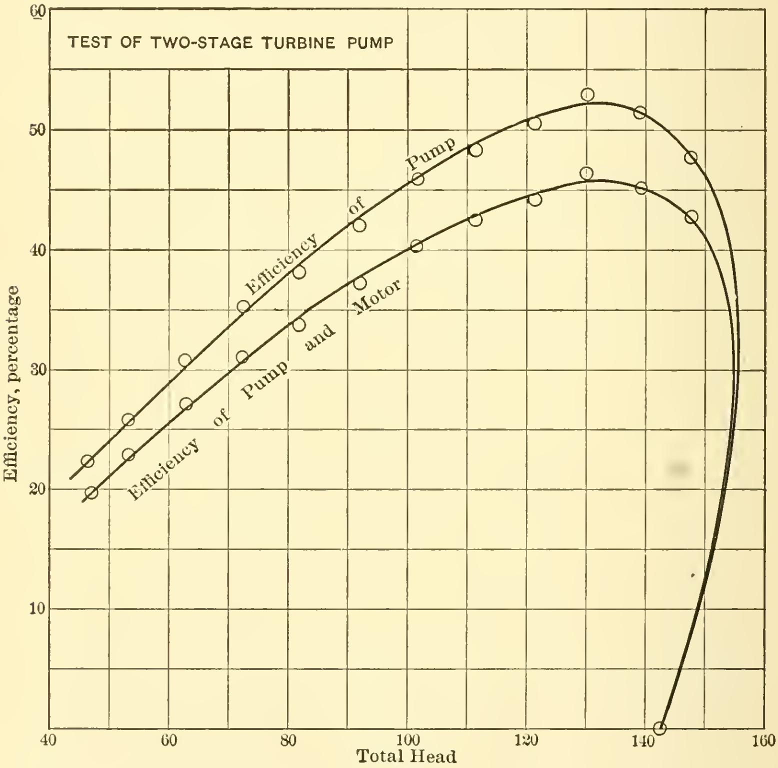

Fig. 5.

Such pumps are not at all common; they are the exception and not the rule. From 60 to 65% represents more nearly the average case, and it is not at all difficult to find examples which fall much below these figures.

Tests of pumps used in hydraulic dredging were made by the Mississippi River Commission in 1902 and 1903. The total efficiencies were found to range from about 57 to 68% in the various

cases. These results are very good, and will compare with those obtained from many centrifugal pumps using lower velocities. The pumps were large, and the diameters of the discharge pipes varied from 32 to 36 in. Mr Gregory.

Mr. Dunham. H. F. Dunham, M. Am. Soc. C. E.—If the speaker is not mistaken, the author has failed to mention the size of the centrifugal pump or the diameter of the rotor. Dimensions and capacity are so closely related to the percentage of efficiency obtainable that it would be well to include them when possible. The speaker's rather limited acquaintance with centrifugal pumps and their manufacture does not coincide very closely with the author's experience. The manufacturer usually knows pretty accurately, from his own shop tests or published tests, the efficiency of the pumps that he puts upon the market, and, while now and then an anxious salesman or a man not anticipating any check upon his statements may be extravagant to the degree indicated, yet the speaker believes that generally, in the East, it would be difficult to find a manufacturer who claimed 50% for a small centrifugal pump. When the capacity is much increased—perhaps to a No. 7 or No. 8, in which the number indicates the diameter of the discharge, and the working head is not very great—there might be some assurance from reliable manufacturers of an efficiency approaching 50 per cent. It would be of interest to know whether the maker of the pump in question first put it out with that guaranty of efficiency, or whether the guaranty was made by some agent unacquainted with facts usually known to the builder, who was thus drawn into the discussion and led to "take chances" upon the result. The speaker recognizes a wide difference between ordinary oral "claims" for efficiency and the incorporation of such "claims" in the terms of a contract.

Mr. Harroun. Philip E. Harroun, M. Am. Soc. C. E. (by letter).—The writer desires to correct a misunderstanding by Mr. Richards. The test of the unit was in no sense an effort to educate the engineering students of the University of California, but was simply a test of a piece of commercial apparatus for the purposes indicated in the paper. The result of this test, together with the results of other tests made by Professor Le Conte, will be published ultimately by the United States Department of Agriculture.

All Mr. Dunham's inquiries are answered specifically in the text of the paper, with the exception of his question relating to the size of the rotor. Detailed prints showing the construction and dimensions have not been available at any time.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

A NEW GRAVING DOCK AT NAGASAKI, JAPAN.

Discussion.*

By L. F. Bellinger, M. Am. Soc. C. E.

Mr. Bellinger.

L. F. Bellinger, M. Am. Soc. C. E. (by letter).—Mr. Hollyday states that practically no money has been spent for repairs on the body of Dry Dock No. 1, at the New York Navy Yard; but, as a matter of interest, it is well to note the fact that the entrance to that dock was practically rebuilt some years ago by P. C. Asserson, Civil Engineer, U. S. Navy. On account of quicksand, the old entrance to the dock began to bulge upward. The rebuilding consisted of an inverted granite arch. This work was completed about August 1st, 1888, the total cost being about $86 000.

The repairs mentioned as being novel by Mr. Hollyday were completed under his direction, therefore, it would be of considerable interest if he would give the Society a detailed account, especially as they pertained to the injection of cement into sand to serve as a cut-off for water percolation, a subject on which information has been requested in the technical papers without eliciting any replies.

* Continued from February, 1906, *Proceedings*. See October, 1905, *Proceedings* for paper on this subject by Naoji Shiraishi, M. Am. Soc. C. E.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THE POSITION OF THE CONSTRUCTING ENGINEER, AND HIS DUTIES IN RELATION TO INSPECTION AND THE ENFORCEMENT OF CONTRACTS.

Discussion.*

By Albert J. Himes, M. Am. Soc. C. E.

Albert J. Himes, M. Am. Soc. C. E. (by letter).—It is a pleasure for the writer to express his appreciation of the kindly reception which has been accorded to a paper which, both in subject and in character, differs materially from those generally presented to the Society for discussion. He fully believes that the subjects discussed are of the most vital interest to all constructing engineers, and can be studied exhaustively with much profit. Mr. Himes.

Mr. Russell's suggestion that "the broad question of the proper interpretation of engineering specifications * * * should be brought up annually for discussion" is very pertinent, and is deserving of careful consideration.

The industrial development of the country has been so rapid that customs affording a sufficient basis for common law have hardly had time to mature; surely not time enough to meet the old English requirement that they shall have existed from "a time whereof the memory of man runneth not to the contrary." It should be the aim of the engineer, and especially of this Society, to work for the establishment of such customs, and thereby purge the profession of all

* Continued from February, 1906, *Proceedings*. See November, 1905, *Proceedings* for paper on this subject by Albert J. Himes, M. Am. Soc. C. E.

Mr. Himes. loose practices and slovenly methods which may lower it in public esteem or hinder it from becoming a stronger force in the social order.

The establishment of a court or "contract committee," as suggested by Mr. Smith, to pass upon disputed questions would be an ideal way in which to establish such customs. That is precisely the way in which the common law was originally developed, and is an eminently proper way in which to develop a common law that may be applicable to modern conditions. That such a court would have no legal status, is not material, for there is a large class of cases in which the parties only wish to be assured that they are treated fairly and according to custom. It is the suspicion of unfair dealing that breeds the larger number of disputes, and the decisions of such a court would be more satisfactory in their adjustment than the rulings of a court at law.

Although the decisions of the court would have only the force conferred upon it by the applicants, in time there would be established a series of precedents which could not be overthrown by the law itself.

Such a procedure is not without example, other than the original development of the common law. The British and North German Lloyds had a similar origin, in the association of vessel owners for the protection of their mutual interests, and the greatness of their success exceeds the wildest flight of the imagination.

An example of the work which such a committee might hope to accomplish may be found in the engineering custom of paying only for materials actually used in the work. The custom in vogue among builders and architects of measuring quantities according to various arbitrary rules is chaotic and illogical, and is distinctly inferior to the engineering method.

Mr. Aiken has mentioned some of the sophistries used by manufacturers in excusing their products. Such methods, to one who understands them, appear to be silly, but they cause lots of trouble among people who are unfamiliar with the business. Only an experienced man knows the value of experience, and it can hardly be expected that a dry-goods merchant will be interested in the phosphorus content of the steel for his new store building, or that a chief engineer who never saw a steel mill will take much interest in rail inspection.

Mr. Haring has related some of his troubles, and, perhaps, he at least will excuse the writer for referring to one of his own. There were large quantities of slope wall to build on three divisions, over one of which he had nominal control. The specification was impossible of execution, and he issued instructions to his assistants explaining what should be required. For this he was severely criti-

cised, it being held that any such "let up" would afford an excuse Mr. Himes. for ignoring the specification entirely. After the work had been in progress about six months, his chief took him to task for exacting such good work of the contractors, saying that the men in charge of the other divisions were not so particular. It is plain that, by using a little reason in the first case, good work was obtained, whereas, on the other divisions, in attempting the impossible, the result was a failure, and the work was as bad as it well could be. The solicitude of the chief for the welfare of the contractors was the cause of a change of management.

The opinion of Mr. Lovell, on the use of a blanket clause in the specifications, is commended to the profession. A man who does not know or cannot describe what he wants should get some one else to describe it for him.

Mr. Thompson emphasizes the need of uniformity of practice. This seems to be one of the aims of the United States Reclamation Service, and the writer has felt the need of it in his experience. The American Railway Engineering and Maintenance-of-Way Association is doing some excellent work in this direction, and its influence will be strongly felt in the profession.

Mr. Beahan's discussion has made very clear the meaning of a contract and the correct attitude of the engineer. His words should be studied with care. In them the young engineer may find more of value than in a whole library of technical books.

To Mr. Bixby, the writer is especially indebted for his kindness in giving to the Society some of his experience as an attorney in cases involving engineering work. His estimate of the value of a diary showing the progress of a piece of construction should be impressed upon every mind. He has also made it clear that an engineer is not a judicial officer, but that the value of his decisions rests on his expert knowledge and in his faithful obedience to the law which requires that his decisions shall be reasonable. In other words, his authority exists almost entirely because of his knowledge and integrity. If he is lacking in these, his decisions will not stand.

In summing up the discussion, the conclusion that seems to be most strongly impressed upon the mind is that, after having studied mathematics and the applied sciences, after having learned to run a transit and become an expert workman, if a man would attain eminence as an engineer, he must descend to the level of a newsboy and study human nature. He must begin at the bottom and seek to understand the ways of men, as individuals and as social units. He must acquire a knowledge of the laws of society and adjust himself so that he may move and act in harmony with its highest aims. Until he does this he is like a pebble in a pot-hole forced around and around by the rushing torrent. When he finally discovers his true

Mr. Himes. position, when he discerns his correct relations to other beings and his surroundings, then, like an atom in Nature's chaos, he may select his affinities, unite in harmony with others and crystallize into a bulwark of society against which the tide may surge and the storms may beat, but over which they will not prevail.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THE ECONOMICAL DESIGN OF REINFORCED CONCRETE FLOOR SYSTEMS FOR FIRE-RESISTING STRUCTURES.

Discussion.*

By Messrs. Wilbur J. Watson, Clarence W. Noble, I. Kreuger, Richard T. Dana, C. A. P. Turner, Ernst F. Jonson, Leonard C. Wason, E. P. Goodrich and Edwin Thacher.

Wilbur J. Watson, M. Am. Soc. C. E. (by letter).—Many of the larger cities have building codes which prescribe certain assumptions which must be made in computing reinforced concrete building work, and some of them require that copies of the computations shall be filed with the city authorities. Under such restrictions, the engineer is not at liberty to exercise his own judgment as to the formulas which he shall use, but must select those which will give rational results and still comply with the requirements of the building code, which are often irrational. One of these sets of requirements is as follows: Mr. Watson.

First, that the ratio of the modulus of elasticity of steel to that of concrete shall be taken as equal to 12;

Second, that the stress in any fiber of concrete shall be assumed to be directly proportional to its distance from the neutral axis;

Third, that the unit stress in concrete, due to bending, shall not exceed 500 lb. per sq. in.;

* This discussion (of the paper by John S. Sewell, M. Am. Soc. C. E., printed in *Proceedings* for December, 1905), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Communications on this subject received prior to April 27th, 1906, will be published subsequently.

Mr. Watson. Fourth, that the unit stress in steel shall not exceed 16 000 lb. per sq. in. under bending loads.

The writer does not wish to be understood as defending the foregoing requirements, especially the first two, which are shown by the author, and also by George H. Blakeley, M. Am. Soc. C. E., in his excellent analysis of the subject published in *The Engineering Record* of May 27th and June 3d, 1905, but is discussing the matter from the viewpoint of the practical designer who is compelled by law to conform to those restrictions. In proportioning the quantity of steel, the writer uses a value of 0.85 of the distance from the top of the beam to the center line of the steel reinforcement as the effective depth of the girder, as recommended by the author. The writer does not like the idea of using multiplied stresses in the analysis of reinforced concrete beams, as it seems to be more rational, and more consistent with the method of analysis used for other types of beams, to use the working stresses assuming the modulus of elasticity of concrete to be constant under the ordinary working stresses. It is often argued, in favor of complicated formulas for designing reinforced concrete beams, that the tests of beams show a greater strength than the more rational and simple formulas would indicate, but such is also the case in regard to the ordinary steel-plate girder. Most writers who deal with this subject place too much reliance on the ultimate strength of a reinforced concrete beam.

It is pretty well demonstrated by tests that, when the stress in the steel rods reaches a comparatively low value, cracks will begin to appear on the tension surface of the concrete. It seems to be the idea that it is safe to disregard this cracking of the concrete in tension, entirely on the supposition that, provided the stress in the steel does not exceed the yield point or elastic limit of the same, these cracks will close up on the removal of the load, and no harm will be done to the beam. Now, the writer is a skeptic on this point, and, until much more definite data are obtainable, relating to the cracking of such beams on their tension surfaces, and the consequent exposure of the reinforcing rods to corrosive influences, it would seem to be wise for conservative designers to use low unit stresses in the reinforcing steel, in order to reduce, as far as possible, the strain of the concrete in tension.

Granting that it is advisable to use a low unit stress in the steel reinforcing bars, there appears to be very little advantage to be gained in using a high-carbon steel, or, in many cases, a distorted bar. The only advantage that appears to be gained by the use of high-carbon steel is the increase in the factor of safety obtained, provided that such factor of safety be based upon the ultimate failure of the beam. If, however, the factor of safety be based upon

Mr. Watson. the point at which damaging cracks occur in the tensile surface of the beam, the factor will be the same for soft steel as for high steel bars. It will be found that by using a low unit stress in the steel and by using small-sized bars in order to obtain a large adhesive surface, it is often unnecessary to use distorted bars, as conservative values of the adhesion of the concrete to plain bars will suffice to transfer the web stresses to the bars. The superiority of soft or medium steel bars over high carbon bars lies in the greater reliability of the former and the fact that they may be safely bent cold in the field. The writer is firmly of the opinion that when high carbon bars are used, as they very often are, they should never be allowed to be bent at all, as they are very likely to break in the bending, if bent cold, or to be improperly treated, if heated and bent. The writer knows of several cases where high steel bars have repeatedly broken while being bent for use in reinforcing concrete work. If, however, high carbon steel is used, and also high working stresses, then it will often be found necessary to use distorted bars, as in many cases the adhesion of the concrete to the steel will not then suffice to transfer the higher stresses. The writer is not quite convinced of the necessity of attaching web members rigidly to the horizontal bars. The method practiced by the writer has been to use vertical U-bars which are placed in the forms first, the horizontal bars are then placed upon them and are supported by them, the U-bars being hung at their upper ends upon a longitudinal bar supported by blocks placed upon the floor forms. In this way the vertical rods are drawn up tight against the horizontal bars and are thus enabled to transfer tensile stresses directly to the latter. As the U-bars are held by the concrete from slipping along the horizontal bars, the writer cannot see why this is not as good as a rigid attachment to the horizontal bars. The system is very convenient in the field. The writer agrees with the author that web members should be used in nearly all cases of beams, and especially when the beams and girders are placed and allowed to set before the floor slab is laid, which seems to be a common method of procedure.

Mr. Noble. Clarence W. Noble, Assoc. M. Am. Soc. C. E. (by letter).—The writer has read this able paper with a great deal of interest. It is particularly opportune, as the use of reinforced concrete is becoming very general, before many of the theoretical points connected with its design have been definitely decided, and before engineering practice in this regard has been in any degree standardized. For example, there is in common use a bending moment formula which gives an ultimate value for a given beam 100% in excess of the value given by another formula also generally used, and both formulas are sanctioned by practice. The variation in practice regarding the use of shear bars is another case in point. It is to be hoped, therefore, that this paper will be very fully discussed.

Mr. Noble. Some time ago the writer, by graphical methods, reached the conclusion derived analytically by the author, namely, that economical design requires that, unless prevented by other considerations, the steel reinforcement should develop the crushing strength of the concrete when its own elastic limit is reached. Just what percentage of steel is necessary to develop this crushing strength is a question on which opinions differ. A. N. Talbot, M. Am. Soc. C. E., found that in 1-3-6 concrete beams 60 days old the crushing strength of the concrete would not be developed by less than 1½% of mild steel or 1% of high-carbon steel. W. K. Hatt, Assoc M. Am. Soc. C. E., found that 2½% of mild steel did not develop the crushing strength of a 1-2-4 rock concrete beam loaded centrally. Edgar Marburg, M. Am. Soc. C. E., found that a beam reinforced with 1.19% of Ransome bars having an elastic limit of 58 000 lb. per sq. in. did not fail by the crushing of the concrete. These tests show allowable percentages of steel considerably in excess of those derived analytically by Captain Sewell. On the other hand, A. L. Johnson, M. Am. Soc. C. E., working analytically, reached a result slightly below those under discussion. Theory and practice here seem to part company. As the maximum of economy is attained in beams highly reinforced, this is a point concerning which investigation can be profitably continued.

Economy in the choice of materials is a matter which is pertinent to the subject under discussion. In designing roofs, for example, it is frequently found theoretically possible to use slabs much thinner than the 3-in. limit fixed by good practice. In such cases, using the 3-in. slab, if the reinforcement used is less than 0.5 of 1% of high-carbon steel it is advisable to use cinder concrete, as the material is cheaper and affords sufficient strength. The choice between the various forms of patented and unpatented reinforcing bars also resolves itself into a question of how much total elastic limit in a satisfactory form can be bought for a cent. If a given amount of tensile strength, together with the necessary bond between the concrete and steel and sufficient provision for shearing stress, can be supplied by plain bars at less expense than by patented bars, then, obviously, the cheaper bars should be used. This brings up a discussion of the nature of the bond between steel and concrete.

The most extensive series of tests of the union between steel bars and concrete, known to the writer, was made at the Massachusetts Institute of Technology.* The report of these tests has also been widely circulated by the patentees of a certain kind of deformed bar. A number of plain and deformed bars were embedded for various lengths in 1-3-6 concrete blocks. These bars were pulled out by direct tension from an Olsen testing machine. The bars

* *The Railroad Gazette*, September 18th, 1903.

extended entirely through the concrete, and observations were taken on the free end to determine the first slip. The tests on round and square plain bars of structural steel are shown in Table 1. Mr. Noble.

TABLE 1.—TESTS OF ADHESION BETWEEN PLAIN BARS AND CONCRETE.

Size and kind of bar.	Depth embedded, in inches.	Adhesion, in pounds per square inch exposed to concrete.	Stress per square inch in bar.
¾-inch, round	24	271	38 400
¾-inch, square	24	274	35 200
¾-inch, round	31	255	42 200
¾-inch, square	31	243	40 400
¾-inch, round	36	219	42 200
¾-inch, square	36	221	42 700

It will be noted, from Table 1, that in every case the bars slipped in the concrete at a point where the steel was strained to more than its elastic limit. This makes an obviously unfair test, for, when the bar begins to elongate, the entire load is taken off by the concrete immediately around the point where the bar enters it, and consequently the adhesion between the concrete and that portion of the bar farther within the block will not be stressed. The resultant action would be similar to that in tearing a piece of paper, which can withstand a considerable tensile stress applied uniformly over the sheet, but fails at once when the load is concentrated at one edge. Such an action shows very plainly in the tests given in Table 1, as the longer bars failed at considerably lower average adhesive stress per square inch than the shorter bars. This condition probably exists to an extent in tests made within the elastic limit of the steel, although it would never occur in a reinforced concrete beam under actual working conditions, as there the stress is applied gradually by the concrete acting on the bar, and the ends of the bar are unstressed at all times. Overlooking this fact, however, Table 1 would show that one can expect an adhesion between concrete and plain steel bars of more than 275 lb. per sq. in. Professor Hatt finds values varying from 636 to 756 lb. per sq. in., and states that, after the rod starts, from 50 to 70% of its original adhesion remains, due to the grip of the concrete. Bauschinger obtains values ranging from 570 to 640 lb. per sq. in. If experimental tests are of any value, an assumption of 250 lb. per sq. in. for adhesion is certainly conservative. This being the case, the statement made by the author that the sum of the horizontal components of the stresses in the web members on each side of the center of the beam should be sufficient to develop the strength of the flange reinforcement, can

Mr. Noble. only be justified as economical practice on the ground that the bond between the steel and the concrete below the neutral axis of the beam is unreliable. He must also consider the ability of the concrete to resist shear as worthless. The majority of engineers dealing with reinforced concrete will probably not agree with the author on these points.

J. W. Schaub, M. Am. Soc. C. E., talking before the students of the Armour Institute of Technology, compared experiments with rods embedded in concrete with those made with concrete pats allowed to set upon steel plates. He concluded that the adhesion between concrete and an embedded rod is due to two causes. The first is the formation of a slightly soluble silicate of iron having a cementitious value in tension of 22 lb. per sq. in. The second is the gripping effect of the concrete due to shrinkage during setting, which pushes particles of cement and sand into the minute unevennesses of the surface of the bar. The silicate of iron causes that portion of the adhesion found by Professor Hatt to be lost with the initial slip. Assuming either Hatt's or Bauschinger's values as correct, all the silicate can be dissolved out of the concrete, or can attach itself to mill scale instead of to the steel itself without reducing the adhesion to 250 lb. per sq. in. Evidently, therefore, this can be assumed as a safe ultimate value.

The advocates of deformed bars advance the argument that the lengthening of a bar under tension tends to decrease the diameter of the bar and thus relieve the grip of the concrete. Now, it is not at all certain that the lengthening of the bar within the elastic limit will cause a corresponding loss of diameter. Certainly, the tendency is to separate the molecules of steel forming the bar, and it is hard to see how this tendency can be transmitted from molecule to molecule along the length of the bar unless some separation actually takes place. But, assuming that the diameter is reduced correspondingly, and that a high-carbon bar having an elastic limit of 50 000 lb. per sq. in. is stressed from zero up to that limit, it would only be increased by $\frac{5}{2900}$ of its original length. A corresponding reduction of area would mean that the reduced radius would be 0.99914 of its original length, thus reducing the radius of a $\frac{1}{2}$-in. bar by 0.000215 in. and the radius of a 1-in. bar by 0.000430 in. If it be assumed that the concrete grips the bar with a pressure of only 250 lb. per sq. in., and that the modulus of elasticity of concrete is 2 400 000, it would only relieve the compression for a distance of 2 in. from the $\frac{1}{2}$-in. bar if the concrete should follow the receding steel for the entire distance of 0.000215 in. Evidently, there is no danger that the reduction in diameter of the bar will cause the particles of the concrete to withdraw from the interstices of the steel surface.

A refusal to recognize the shearing value of concrete can only arise from the belief that when the beam has taken an appreciable deflection, so that the concrete below the neutral axis has been separated by minute cracks, this shearing value disappears. This is doubtless the case where such cracks occur, but, taking place as they do at points where the steel is stressed the highest, they occur at points of lowest shear, and, even here, unless the steel is loaded to more than its elastic limit, do not go above the neutral axis of the beam. Even if the beam is designed for a concentrated moving load, the concrete above the neutral axis near the middle of the span would be as well able to take the maximum shear coming on it as would the concrete in the full depth of the beam near the ends. Therefore, it would seem to be only necessary to provide vertical reinforcement for shear in excess of that which can safely be carried by the concrete. Mr. Noble.

It is the writer's belief that economical designs are best obtained by the use of a comparatively large number of small undeformed bars for horizontal reinforcement. This insures ample bond between the steel and the concrete for all but very short and deep beams. As the ends of the beam are approached, the flange stresses become of relatively less importance, and these bars are then, one or two at a time, turned up at an angle of from 30 to 45° into the concrete and continued along the top of the beam to the point of support, where they bond into the wall or adjacent beam. They thus serve successively as positive-moment bars, shear bars, and negative-moment bars, meeting in each situation the greatest need of the beam, and doing it always with a maximum of economy.

I. Kreuger, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Sewell gives some very valuable suggestions for the design of reinforced concrete beams. The writer believes, however, that until more thorough knowledge of the properties of concrete is obtained, the straight-line formula is the most satisfactory one for computing reinforced concrete girders. Mr. Kreuger.

There is hardly a prominent engineer in Europe or America dealing with concrete who has not established a formula of his own, usually built upon an assumed variation in the stress-deformation curve of the concrete. The factors which influence this curve seem to be so many and of such a complex nature as to make hopeless the task of obtaining a result, true for all classes of concrete. While numerous observations as to the deformations of concrete under compression have been made in Europe and America, the widely different results obtained tend to make them rather confusing, and, under these conditions, the author's stress-strain curve seems to be an unwarranted refinement.

The simplicity of the straight-line formula, and the fact that it

Mr. Kreuger. is accepted for the building laws of many prominent cities, are factors greatly in its favor. It seems to the writer, however, to be of secondary importance to establish the true stress-strain curve of the concrete in compression, as all tests show that this curve is quite different from the stress-strain curve of the concrete subjected to bending. This is not always clearly recognized, but the customary provision to calculate a higher value for bending compression than for direct compression is an admission of, and an allowance for, the errors of the present method of computing reinforced concrete girders.

Such evidence can also be found in the tests mentioned on page 652* where the steel was pulled apart, though the percentage of steel was more than twice the amount recommended in the table on page 634.*

If one insists upon computing concrete girders in a manner similar to that used for steel beams, and if one maintains the assumption that the tensile strength of concrete should be neglected, the results of the bending tests executed on reinforced concrete girders would lead to the assumption of a stress-strain curve following closely a rectangle. The diagram for such a curve would then have the appearance of Fig 7.

The writer, however, believes that, until more thorough tests have been made on this subject, the straight-line formula at present in use is the most convenient way for calculating reinforced concrete beams.

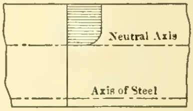


Fig. 7.

The conclusions to which the author comes, that for economical design as large a percentage of steel as the concrete will stand should be used, and that no steel should be put into the upper flange, seem to be thoroughly sound, and are probably in conformity with the experience of most designers.

Regarding T-beams, the author brings up the question of the flange width which should be counted on. The praxis on this point varies greatly. The building laws of several cities, notably Cleveland and Buffalo, allow a width of ten times the width of the beam. It is contrary to all authorities on this subject, however, to use such a large flange width, and the figure given by the author—three times the width of the beam—seems to be much more satisfactory. It is difficult, however, to see in what way the allowable flange width is influenced by the width of the beam, and it would seem to be more correct to make it dependent upon the length of the span. For certain cases where there is danger of buckling, the thickness of the slab may be the governing factor. It has been the writer's praxis to allow a total flange width of one-sixth of the length of the beam.

* *Proceedings*, Am. Soc. C. E., for December, 1905.

The system of web reinforcement described by the author is a very interesting feature of the paper. It may be said that the value of web reinforcement has been clearly established, and most engineers agree upon this point. Mr. Kreuger.

Of the author's final remarks, the writer considers No. 6 particularly worthy of attention. If concrete girders were generally calculated as continuous, and the steel reinforcement arranged accordingly, a great saving could be made, and at the same time there would be a gain in safety.

Richard T. Dana, Assoc. M. Am. Soc. C. E. (by letter).—The author has taken the longest step, so far, toward placing the design of concrete structures on a strictly scientific basis. The mathematical work is admirable, and the deductions, in the main, seem to be accurate. The writer does not entirely agree with the author in placing the cost of steel at 3 cents per lb., as it would seem that $2\frac{1}{2}$ cents would be a closer figure, bringing the average value of p to 60. The author has designed his formula for ultimate strength. The writer is of the opinion that it is better practice to design for working stresses, for the following reasons: Mr. Dana.

1.—A very much smaller part of the stress-strain curve is brought into play, and to the stress-strain curve a close approximation may be made by a straight line, thus permitting the use of a straight-line formula.

2.—The factor of safety for concrete should, at times, be decidedly different from the factor of safety for steel in tension. If a beam is designed on the basis of the point of failure of the steel and concrete, or on the basis of the point of failure of the steel, namely, the elastic limit of the steel, and a percentage, say, 80, of the point of failure of the concrete when the beam is under vibratory loads, the concrete will be working under a disadvantage, as compared with the steel, and therefore the concrete will be overworked, which would result ultimately in weakening the beam. It would seem to the writer, therefore, to be better practice to design for the actual safe stresses of the two materials.

The formula worked out by the author is exceedingly ingenious, and is the simplest one known to the writer, having regard to rigidity of reasoning. In beam formulas, however, the writer prefers to use the method delevoped in the last few years in France by Coignet, Tedesco, and Maurel. The method replaces the steel in a section with a quantity of concrete equivalent to the area of the steel multiplied by $\frac{E_s}{E_c}$. The section, thus reduced to a homogeneous one, is referred to a neutral axis passing through the center of gravity, and the moment of inertia of the entire section is obtained on the basis of the concrete alone. This method results in extremely simple

Mr. Dana. formulas for rectangular T-beams, and is applicable to the rapid analysis of very complicated shapes. It is the only rapid method known to the writer whereby a hollow section of irregular area, with various arrangements of reinforcement, can be calculated accurately. The value of N, or the ratio between the coefficients of elasticity of steel and concrete, will, of course, vary for different conditions, and it seems to the writer to be essential, in any general formula, that N be chosen by the designer for the special conditions of practice, and that it should be in shape to insert readily in the formula.

Concerning the question of web stresses, in the writer's opinion, the concrete beam is analogous, not to a Pratt truss, or to a Warren girder, but rather to a Howe truss. While the method of reinforcement given by the author is unquestionably excellent, and will produce a safe beam, it is believed by the writer that an equally safe and rather cheaper beam can be made with vertical stirrups spaced at distances equivalent to the depth between the centers of tension and compression, with special attention paid to what in a plate girder would be end stiffeners.

With the practical reasons, given by the author on pages 654 and 655,* for the use of stirrups or web members, the writer fully concurs, except in the argument based on the reconstruction of such beams in place. Where the heat from the fire has been so great as to dehydrate the cement, the steel is likely to have suffered greatly, and even if the concrete were replaced, the beam would probably not have anything like its original strength, particularly if cold-drawn wire or cold-twisted steel were used for the reinforcement.

The thanks of the engineering profession, as well as of this Society, are due to the author for this admirable paper.

Mr. Turner. C. A. P. Turner, M. Am. Soc. C. E. (by letter).—While the writer has designed a great many buildings of reinforced concrete, and while, in every case where his instructions have been carried out, the resulting work has been stronger than represented by him, he will frankly state that though he has been able to design safe and satisfactory work at a low cost for construction, he regards it as impossible in the present embryonic state of knowledge of the properties of reinforced concrete to attempt successfully anything approaching a valuable general mathematical discussion and investigation of the economical design of reinforced concrete floor systems. This statement is made in fairness to Mr. Sewell in order that such criticism as the writer offers may be better understood, and that such theories as are presented with the criticism will be accepted in their true light, as a merely suggested explanation of observed facts.

It happens too often in a mathematical discussion that the

* *Proceedings*, Am. Soc. C. E., for December, 1905.

FIG. 1.—WAREHOUSE, N. W. KNITTING CO., SHOWING CONSTRUCTION WITH 5¼-IN. SLABS, 16 FT. 8 IN. BY 15 FT. 8 IN.

FIG. 2.—WAREHOUSE, N. W. KNITTING CO. TEST LOAD OF 900 LB. PER SQ. FT. ON 5¼-IN SLAB, 16 FT. 8 IN. BY 15 FT. 8 IN.

theorist starts out with an assumed premise, then proceeds to build an elaborate mathematical theory thereon, and forgets in his summing up that his reasoning, however accurate his mathematics, is, after all, based on the assumed premise rather than on fact. This seems to the writer to be the basis of the author's expressed belief that, in his short discussion, he has brought out the real principles of economic design. Mr. Turner.

The author's assumptions will be taken up in detail, in order to arrive at the value of his conclusions. First, he asserts that no extensive system of concrete floors can be economically designed without the use of rolled-steel beams or concrete ribs. Now, the writer's experience, in paying the cost of this work in labor and materials, is that where the panels are not greater than 25 ft. square, for a guaranteed test load of 200 or 300 lb. per sq. ft. over the full area, a plain slab without ribs costs less than one with ribs. In warehouse work, it is perfectly feasible to put up a building with columns at 16-ft. centers, with a floor of 7½-in. rough slabs, using no ribs at all, and test it with 800 lb. per sq. ft. without injury to the construction. Furthermore, it can be put up at less cost without the ribs, and will require less metal, as the load will travel more directly to the supports, instead of around a corner, as in the case where beams are used. This method of construction is outlined in Figs. 8 and 9.

Now, floor slabs are made anywhere from 5 to 25 ft. in span, and, on this basis, can a discussion, of the economic relations of the slab and ribs, which starts out by assuming a constant value for the thickness of the slab be regarded logically as of any value whatever? Aside from this little oversight, the author fails to realize that if beams or ribs are at 5-ft. centers, the rib centering costs just four times as much as if they were at 20-ft. centers. This, also, is an item which is not fixed by the problem in most cases, but by questions of economy only, and must appear in the discussion, if it be of any practical value whatever. Hence, the cost of centering, which the author assumes as a constant, is in reality a very variable one. Again, the floor system is but a part of the whole, and, as the same load can be carried to the footing more cheaply through one column than through two, this item enters into the discussion, if it be complete.

Now, in any physical research, a mathematical theory is of value only as it agrees with and explains the results of practical experiment and if the author's theory is tenable, as a general basis of economic design, it should fairly explain the results and facts developed in the course of practical work. Take, for example, the floor slabs illustrated in Plate XXV. The panel tested was 16 ft. 8 in. by 15 ft. 8 in., in which the slab was 5¼ in. thick with 1 in. of cheap strip filling on the top, reinforced with ⅝-in. round bars, at

Mr. Turner. 4-in. centers each way, each kept at an average of $\frac{3}{4}$ in. from the bottom of the slab. As these rods were kept close to the bottom right through, they could not be considered as reinforcing the slab over the beams on the top or tension side, as would be considered in the ordinarily accepted theory. The rods were long enough to go just over the top of the beams and were hooked at the end. A test load of cement in sacks of approximately 900 lb. per sq. ft. was

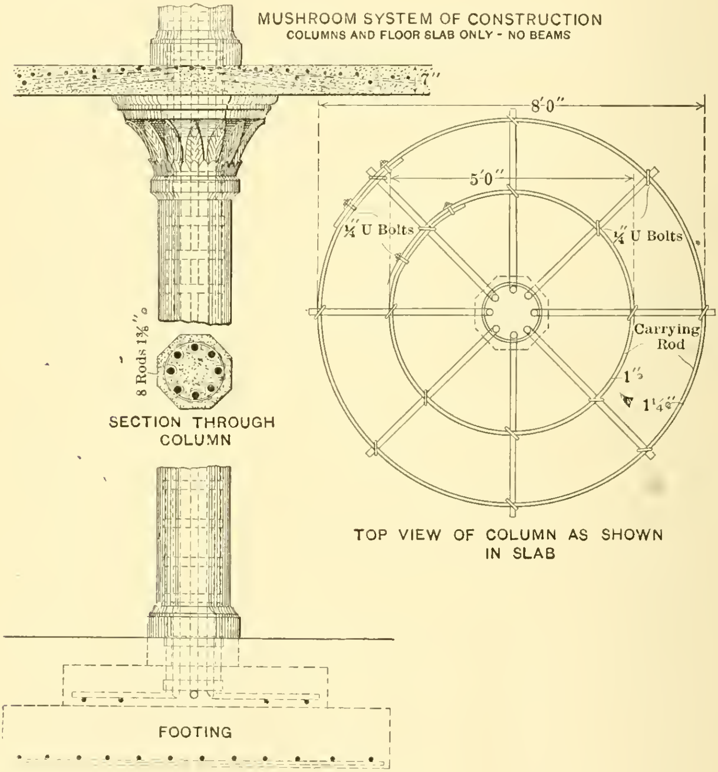
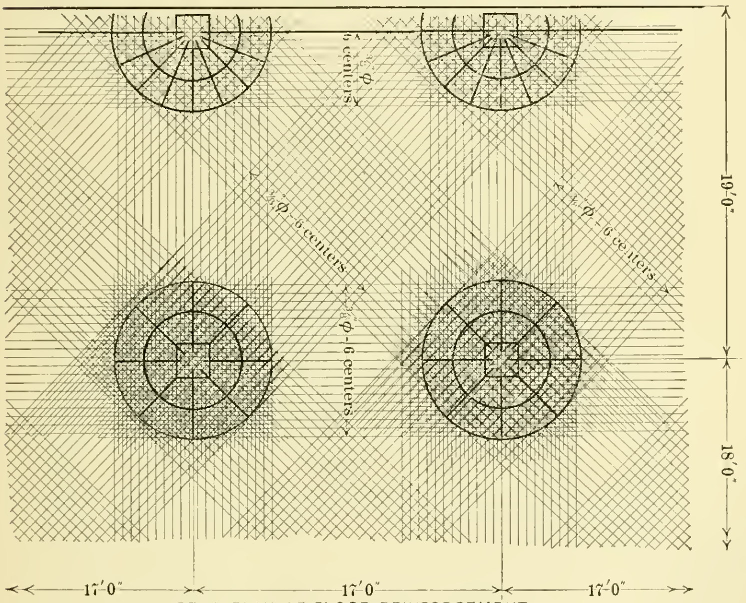


Fig. 8.

applied to the slab and kept inside the beam lines so as to give a straight shear load on the slab. The strip filling was a very weak mixture of lime, cement, and sand, so that it could have added little to the strength. Disregarding the strip filling and calculating the moment at $\frac{1}{8}$ $W L$, or, what is probably more nearly correct, $\frac{1}{10}$ $W L$, in the present case, distributed equally between the two systems of

rods, gives, in the author's formula: $M = h\,d\,a\,b\,t_s = 0.846 \times 4\frac{1}{2}$ in. Mr. Turner. $\times \dfrac{0.333 \text{ in.}}{12} \times 12$ in. $\times$ 40 000 = 50 760 in-lb. for the ultimate strength. Now, the moment of the load, $M = \frac{1}{10}$ (450 $\times$ 15) 16 $\times$ 12 = 129 600 in-lb.; in other words, the moment of the applied load is 2.6 times the yield point value of the steel, as indicated by the author's formula, leaving the dead weight to care for itself, or be carried by the cement. The actual deflection of the slab, under the

MUSHROOM SYSTEM OF CONSTRUCTION.
COLUMNS AND FLOOR SLAB ONLY - NO BEAMS.



PARTIAL PLAN OF FLOOR REINFORCEMENT

Fig. 9.

test load at the center, was only $\frac{5}{32}$ in., and it would require a deflection of $1\frac{1}{2}$ in., at least, even to crack such a slab. From this fact, it is a fair inference that the strength of the construction was at least from 700 to 800% of what might be expected from the author's economic theory; and either excessively strong concrete is being made or the author's theory is very weak indeed. It should be noted that the concrete tested was only about 7 weeks old, and, probably had not developed more than 70% of its ultimate strength,

Mr. Turner. due to the slow drying at that season of the year; further, that the deflection of the beams was $\frac{1}{16}$ in., leaving $\frac{3}{32}$ in. deflection of the slab between the beams.

Mr. Marsh, in his work on reinforced concrete, Part V, writes as follows:

"It may be that we are wrong from the commencement in attempting to treat it (reinforced concrete) after the manner of structural ironwork. * * * The molecular theory, *i. e.*, the prevention of molecular deformation by supplying resistances of the reverse kind to stresses on small particles, may prove to be the true method of treatment for a composite material such as concrete and metal."

It seems to the writer that the difficulty with the author's theory is in the fact that he is endeavoring to treat the dual material, as

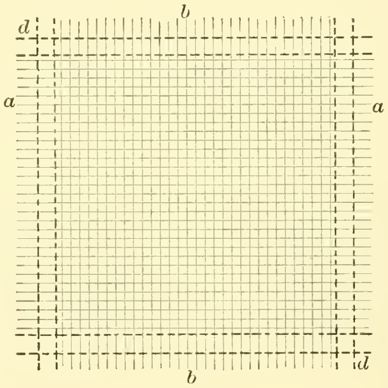


FIG. 10.

Mr. Marsh says, after the manner of structural ironwork. He practically starts out with the assumption that the concrete can be economically reinforced in one direction only, or assumes, following certain other writers, that where the reinforcement is applied in two or more directions, the same treatment holds; and that the stress has only to be divided between the various systems, and that otherwise, their joint action can be legitimately ignored.

Glancing now at Fig. 10, the following facts appear in evidence under any bending of the slab: The rods, *aa* and *bb*, the reinforcement, are in tension and along the diagonal lines; *dd*, the lower fibers, are also in tension. Now, these forces must be balanced by compression in the upper part of the slab. Where the reinforcement runs in one direction only, this stress is cumulative toward

the center, but where it is in two or more directions it may seemingly be carried largely by lateral arching. See Fig. 11. Mr. Turner.

Again, there is little exact knowledge as to how great an increase in strength may be obtained under the conditions that one compressive stress tends to balance the deformation of another. M. Considère's researches have thrown some light on this, and are, perhaps, an indication as to what may be accomplished along analogous lines. It may well be that the expert physicist may be able to devise an apparatus to measure the molecular stresses by thermo-electric means, as the writer has succeeded in doing in steel. For such investigation, a concrete of sand and cement only might simplify the problem.

Next consider the question of shear and bars with attached web members, or their equivalent, which the author seems to fancy because of his assumptions of an analogy between a concrete beam and a truss. In the experiments by A. N. Talbot, M. Am. Soc. C.

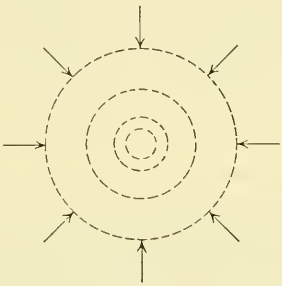


FIG. 11.

E., these bars indicated a strength of only 87% of that of plain bars of equivalent section, and, in view of their irregular shape, rough-sheared and nicked section, it is surprising that they did even as well as that. That good results have certainly been obtained with them, cannot be disputed, nevertheless, 60% of the strength that may be obtained with plain round bars and good work will pass muster anywhere.

As regards the use of shear members, the writer's experience would indicate that better results may be obtained without them, by simply tying in the skin of the beam or rib with a net. The idea of having the flange reinforcement all in the bottom of the beam, except at the center, is certainly rarely followed by those familiar with practical work, and the author's remarks, based on the assumption that the reverse is true, and his conclusions regarding the superior advantages of the attached web member bar, from the fire-proof standpoint, are not substantiated.

Mr. Turner. Comparing the construction by the manufacturers of the attached web member bar with construction where plain bars are used, and referring to the warehouse of the Farwell, Ozmun, Kirk Company at St. Paul as a typical example of the former type, and the warehouse of the Minneapolis Paper Company as an example of the latter, the following tests offer a practical method of judging these claims. In the warehouse of the Farwell, Ozmun, Kirk Company, for the panel tested, the columns were at 13-ft. centers along the main girders, and at 16-ft. centers along the carrying beams, which were spaced at 5.5 to 6.5-ft. centers, and the slabs were about 7 in. thick. The test load was 78 tons of pig iron over one beam, which caused a deflection of $\frac{1}{64}$ in. In the warehouse of the Minneapolis Paper Company the panels were 15 ft. 4 in. by 21 ft. 6 in.; the slabs were $6\frac{1}{2}$ in. thick, over the full panel, with 1 in. of strip filling made of weak mortar; the reinforcement was $\frac{3}{8}$-in. round rods, spaced at an average of 6 in. from center to center each way; the beams ran from column to column in each direction only; the test load was 110 tons (shear load) on the slab, and, later, 50 tons were added to test a 12 by 16-in. beam with 21 ft. 6 in. span. The slab reaction on the beam would be approximately 70 tons. The deflection of the beam was practically inappreciable. Now, if, as above noted, $\frac{3}{8}$-in. rods with an average centering of 6 in. can carry on a $6\frac{1}{2}$-in. slab 110 tons to the beams, at 15 ft. 4-in. and 21 ft. 6-in. centers, the floor slab in the St. Paul warehouse, if the attached web member bar is any good, ought to distribute the load over at least three beams, so that this test was a little greater than the load the floor was designed to carry—500 lb. per sq. ft.—while, in the case of the plain bar design, there was double the load, and on a span one-third longer than that of the building of the Farwell, Ozmun, Kirk Company. There seems to have been in the slab approximately an equal weight of metal reinforcement per square foot in each case, and also per linear foot in the beam, though, in the plain rod design, the clear span of the slab was nearly three times as great, while the beam carried twice the load and was one-third longer in span. Upon such a showing, the attached web member bar would be entitled to rather a scant consideration, if this illustration is a fair one. Figs. 1 and 2, Plate XXVI, show some tests at the warehouse of the Minneapolis Paper Company.

The basis of the author's belief in the web member bar or its equivalent will now be examined critically. He states that he analyzed the web stresses by analogy with a Pratt truss, and, later, at the War College, further study made it apparent that the double-intersection Warren girder was a better analogy. Admitting, for the sake of argument, that it is permissible to draw valuable conclusions from truss construction relative to reinforced concrete beams, ac-

FIG. 1.—WAREHOUSE, MINNEAPOLIS PAPER CO. TEST LOAD OF 110 TONS ON 6½-IN. SLAB, 15 FT. 4 IN. BY 21 FT. 6 IN., CENTER TO CENTER OF BEAMS.

FIG. 2.—WAREHOUSE, MINNEAPOLIS PAPER CO. TEST LOAD OF 160 TONS ON 860 SQ. FT. COLUMN SPACING 15 FT. 4 IN. AND 21 FT. 6 IN., LONGITUDINALLY AND TRANSVERSELY, CENTER TO CENTER.

cording to the author's belief, the Pratt truss was a good analogy and the double Warren girder a better one, but, if an opinion is to be formed in this questionable way, engineers should obtain the best possible analogy. Taking the author's example, the case of the uniform load, and considering an analogy with an inverted parabolic bowstring or deck truss, the reinforcement is of constant section, as recommended by him, the web stresses are all compressive, and the curved reinforcement seems to be ideal in this respect, doing away with his multiplicity of relatively small and bothersome web members, effecting thereby a theoretical economy of 30% of the metal and 75% of the trouble in handling and placing it. This analogical theory, it will be noted, is for uniform loads. There remains, therefore, to treat any combination of live loads in a similar manner for the simple beam. Still proceeding on the author's recommendation of a constant tension chord section, as the compression chord of concrete is constant, one should seemingly look for a suitable analogy in that type of truss which economically discards diagonal panel web members, while complying with the Mr. Turner.

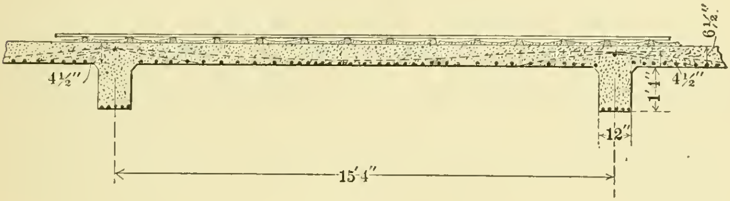


FIG. 12.

above fixed conditions, and this type is evidently represented in the Bollman truss. While, in steel construction, the conditions noted bar it from economical use, with these fixed, it possesses quite a marked advantage over reinforcement planned on the Warren girder order, combined with a dissemination of steel through the concrete and ease in placing it that should be apparent to those having practical experience in this line.

An example of the failure of the theory as applied to a slab which is nearly square has been given, and now the author's formula will be applied to the rectangular slab tested at the warehouse of the Minneapolis Paper Company, 21 ft. 6 in. by 15 ft. 4 in., from center to center, beams 12 in. wide, load 860 lb. per sq. ft., distributed over nearly 13 by 20 ft. in the center of the slab, as shown in Fig. 12.

Weight of construction..........	90	lb. per sq. ft.
Live load.........................	860	" " " "
Total	950	" " " "

Mr. Turner.

Using Mr. Dunn's well-known formula for distribution over rectangular slabs, with continuous reinforcement:

$$M_c = \text{Moment over support} = \frac{W\ B}{12} \times \frac{L^4}{L^4 + B^4},$$

B being breadth, and L length of slab. Therefore:

$$M_c = \frac{12\ 350}{12} \times 15 \times 0.8 = 12\ 000 \text{ ft-lb.} = 144\ 000 \text{ in-lb.};$$

$d = 4\frac{1}{2}$ in.; $h = 0.846$; $t_s = 40\ 000$.

$M = h\ d\ a\ b\ t_s = 0.846 \times 4.5\ (0.44) \times 40\ 000 = 67\ 000$ in-lb. = the calculated ultimate strength, while we have tested it to 2.2 times this amount with a deflection of $\frac{1}{1000}$ of the span, and could certainly apply more than 3 times this load before final failure. It should be noted that the value of a used is the average section, the bars being spaced closer together at the center of the slab, in this particular design, and is taken as double for the lap of the rods in the adjacent panel. One would suppose that this arrangement would show greater stiffness than that first outlined, but a test does not seem to bear out the inference, or, at any rate, to give it a positive standing.

Next, consider the question of the character of the test. Was the loading such that the conclusions are warranted? Was there any considerable arching, and to what extent could the surrounding construction distribute the load? When the ratio of the depth of the slab to its span is only 1 to 8 or 10, as in the building of the Farwell, Ozmun, Kirk Company, the slab can distribute a very material part of the load over adjacent beams, but where this ratio increases from 1 to 8 or 10 to 1 to 25 or 35, the stiffness being approximately as the cube of the span, this amount would be but $\frac{1}{15}$ to $\frac{1}{25}$ of the former, where the ratio is only 1 to 8 or 10, perhaps from $2\frac{1}{2}$ to 4%, calculating this action in the former case, the St. Paul test, as great as 60 per cent.

Now, as to the question of the arching of the load from beam to beam, and reducing the load carried by the slab, it may be stated that the sacks were piled with especial reference to reducing this action to a minimum. With a deflection so small, this action, in any case, would be slight, perhaps reducing the calculated moment less than from 5 to 8%, which might be estimated for a sand or grain pile of these dimensions. It may be fairly said of the load that it is similar to loads that would be applied in use in the building, and that the slab load was piled inside the beam lines to give a straight shear load on the slab. Again, tests have been made on slabs with white lead in kegs, piled so that there was no arching, and the calculated strength, by the formula noted, did not agree more closely with the practical results than in the foregoing instances.

Referring now to the author's conclusions as to the economic

relation of the cost of the concrete and steel, and, taking in this case a plate girder for analogy, it is considered conservative to disregard the web resistance to bending in a plate girder, just as in the concrete beam the web resistance of the concrete below the neutral line is disregarded. Now, according to general practice, it is calculated that the compression flange should equal the tension flange in cost, that the section of the flanges and consequent cost decreases with the increase in depth, and, by principles of maxima and minima, one ordinarily tries to prove the total cost a minimum when the costs of the web and flanges are equal. Mr. Turner.

Now, vary the problem by making the tension flange of a special grade of steel and the web and compression flange of common steel, as before, and write the moment equation thus:

$$M = h\ d\ a\ b\ t \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (A)$$

in which $h\ d =$ the lever arm, or area a of the bottom flange, and $t =$ the unit of tension. The variable quantities of the two different kinds are:

1.—The special grade of steel in the bottom flange; and

2.—The common steel in the web and top flange.

Now let p represent the ratio of the unit costs of the two grades of steel and let x equal a quantity proportional to the sum of the costs of the variable elements, and substitute for a in this equation its value from Equation A, and it can be shown by the author's method that the total cost is a minimum when the cost of the special steel in the bottom flange equals the cost of the other three-quarters of the girder, that is, the common steel web and top flange. It should be noted that the assumed variation of the web and compression flange with the depth is identical with that assumed by the author in the case of the concrete beam.

Taking the first slab illustrated, reducing the concrete, and increasing the steel until the cost of each is equal, $3\frac{1}{2}$ in. is obtained as the depth of the concrete over the steel, and this on a span of 16 ft. 8 in. from center to center. As the steel is medium, h, according to the author's determination, equals 0.85, so that the effective depth, $h\ d$, equals $3.5 \times 0.85 = 2.97$ in., and the ratio of depth to span is as 1 to 65. These would seem to be rather attenuated dimensions to support a load of 900 lb. per sq. ft.; strangely enough, there does not seem to be an excessive percentage of steel, judged by his standards. Again, take the case of a simple beam 10 in. wide and 15 in. deep, the cost of the concrete above the steel is 22 cents, and 22 cents' worth of steel at $1\frac{3}{4}$ cents per lb. in place for plain bars equals $12\frac{1}{2}$ lb., or 3.75 sq. in. This corresponds to 2.75% reinforcement, nearly twice the maximum percentage necessary to develop the strength of the concrete by the author's computation.

Mr. Turner. While, from the engineer's standpoint, the writer can see but little opportunity for the use of this economic theory, from the standpoint of the vendor of reinforcement, if he can but convince the purchaser of its accuracy, the commercial possibilities in this line would seem to be very attractive. This commendable feature of the theory, combined with its apparent plausibility, would readily deceive anyone failing to note the confusion of constants and variables involved in the assumptions on which it is based.

The writer will now examine the assumptions under which the value of h was determined a constant, and see how the author proceeds to fill these conditions in his economic theory. These conditions are that the steel and concrete are each worked to a definite and constant allowable limit. Now, in his economic theory, he proposes to vary d and a to balance a constant moment, M, but, arbitrarily, to keep h and b constants. The writer will start with a depth less than the economic depth and double it, and note at these extremes, under the author's assumptions, the variation of the working stress of the concrete. For the shallow beam, the concrete can be worked, of course, only to its safe working stress. When d is doubled, the total compressive stress on the compression chord (the concrete) is cut in two, b and h remaining constant; by the author's economic theory, the area carrying this stress is doubled, making the working unit stress for the concrete, in the second case, only one-fourth of what it was in the first; in other words, according to the author's assumptions, he compares the economic relations of the steel and the concrete on the basis of a rational and fixed working stress for the steel and, as it happens, an actual variable (within the narrow limits of 300 or 400%) working stress for the concrete. This appears to be rather severe on the concrete, as indicated by the practical example of the slab. Upon such a basis of computation, it is hardly surprising that the author concludes that his theoretical economy, based on relative costs, is not attainable. He also notes that his discussion of minimum cost does not contain the ratio between the allowable maximum stresses in the two materials, but fails to note that this ratio, which should be fixed in a rational discussion, is an extremely variable one as involved in the equations from which he essays to draw his conclusions.

The writer, in his remarks, thus far, has not questioned the author's assumption of a constant section of reinforcement from the economic standpoint. Now, for a simple beam, where the moment varies from zero at the end to $\frac{1}{8}$ W L at the center, the need of the maximum section for the full length is not apparent. In the Moulton Jordon garage there were some short-span girders of reinforced concrete, 42 ft. from out to out, in which the writer considered that there was a very decided economy in designing them with a variable

flange, as he would have done if they had been steel. This saving, of course, would increase largely with the increase in span. Mr. Turner.

In ordinary floor systems, however, there is the general problem of a series of spans, and the question of the relative economy of continuous *versus* simple beams at once arises. As it is impracticable to vary the section of concrete along the length of the beam, and as the maximum moment for the continuous beam for a uniform load is approximately only two-thirds of that for the simple beam (calculating for an intermediate span), the continuous construction should result in a material saving in concrete.

Considering now the steel reinforcement: The moment at the support is double that at the center of the span, and of the opposite sign. Now, the rods are rolled of constant section, and, if they are carried from the tension flange at the center to the tension flange at the support and lapped into the next beam, there would be required, approximately, one-third the section for half the length and two-thirds the section for the remainder that would be required for the simple beam construction, or half the metal required in the simple beam with constant section of reinforcement. With this arrangement, there is the condition that the maximum flange reinforcement is required at the point of maximum shear, that the moment decreases rapidly from the support toward the point of contra-flexure, allowing this main reinforcement to drop sharply downward without weakening the construction, thus placing the main section of metal in a position to carry the entire shearing strain, without counting on the concrete or depending on the questionable amount of adhesion that may be obtained between the concrete and small and stubby web members.

This simple bend in the bars gives them an anchorage in the concrete which, from the writer's experience, appears to discount any form of nicked-section mechanical bond yet invented.

Owing to the fact that the moment at the center is only half that at the support, the question of reducing the concrete further by reinforcement of both flanges at the support, for part of the length only, should be considered, if the economic theory be complete. This method is followed by the writer.

Bad work, in one instance, executed by an incompetent contractor, on a footing, gave the writer an opportunity of judging the amount of distortion a connection of this character would stand, and he was not a little surprised to be forced to conclude that it could stand, if anything, as great an amount of distortion, without material injury, as could be expected from a structural steel frame with standard riveted connections of the web of the beams to the columns. Such reinforcement is more satisfactory from the standpoint of resistance to lateral or vibratory forces.

Mr. Turner. The fewer joints there are in the concrete, the more uniform it is in strength; and any method of placing it piecemeal, as observed by the author to be done frequently, cannot be too strongly condemned. The cement is that part of the composite material which gives it its strength, and, to the largest extent, its fire-proof properties, and anyone who possesses the temerity to follow the author's suggestion of using a cheap concrete for the lower half of a beam, whether with the attached web member bar or any other, is, in the writer's judgment, industriously looking for trouble rather than economy.

In conclusion, it is safe to assert that no one has a higher respect for true theoretical economy than the busy engineer of construction. This brand of theoretical economy is attainable, and is based on a complete and accurate statement of all the facts entering into the problem. That brand which is not attainable, he immediately concludes, is based either on an incomplete and defective statement of the conditions of the problem, or on inadmissible assumptions regarding them.

Mr Jonson. ERNST F. JONSON, ASSOC. M. AM. SOC. C. E. (by letter).—There is one point in Mr. Sewell's paper to which the writer begs to call attention as not being quite correct.

The author takes the depth of the axis of the horizontal reinforcement below the top of the beam as a basis for his shear computation, instead of the depth of that axis below the resultant or centroid of the compressive forces.

The following proposition is true of all beams:

The total shear on any one cross-section of a beam is equal to the average unit shear at the neutral axis, multiplied by the width of the beam at the neutral axis, multiplied by the distance between the resultant of the compressive forces due to the bending moment and that of the tensile ones.

$$S = s\ b\ f.$$

Where $S =$ the total shear on the cross-section;

$s =$ the average unit shear at the neutral axis;

$b =$ the width of the beam at the neutral axis; and

$f =$ the distance between the resultants of the compressive and tensile forces.

The demonstration of this proposition is as follows:

Let $p =$ the longitudinal unit stress due to the bending moment;

$R =$ the resultant of this stress on each side of the neutral axis;

$M =$ the bending moment; and

$a =$ the distance from the neutral axis to the extreme fiber.

It is known that the shear at the neutral axis is equal to the first differential coefficient of the total longitudinal stress due to the bending moment on one side of the neutral axis. Mr. Jonson.

$$s\,b = \int_{x=o}^{x=a} \int_{z=o}^{z=b} \frac{d\,p}{d\,y}\,d\,x\,d\,z \ldots\ldots (\mathrm{I})$$

Hence,

$$s\,b = \frac{d\,R}{d\,y} \ldots\ldots\ldots\ldots\ldots\ldots (\mathrm{II})$$

It is also known that the total shear on the cross-section is equal to the first differential coefficient of the bending moment.

$$S = \frac{d\,M}{d\,y} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (\mathrm{III})$$

And, since $M = R\,f$,

$$\frac{d\,M}{d\,y} = \frac{f\,d\,R}{d\,y} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (\mathrm{IV})$$

Hence,

$$S = \frac{f\,d\,R}{d\,y} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (\mathrm{V})$$

By substitution, according to Equation II,

$$S = s\,b\,f \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (\mathrm{VI})$$

The formula for the maximum unit shear in a reinforced concrete beam will then be:

$$s = \frac{S}{b\,f} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (\mathrm{VII})$$

And the formula for the stress, t, on the 45° diagonal reinforcement in one unit of length will be:

$$t = \frac{S}{f\sqrt{2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (\mathrm{VIII})$$

It seems to the writer that a sharp bend at the junction of the diagonal and longitudinal reinforcements should be avoided, as it tends to produce an excessive compressive stress in the concrete at that point; and that a round bend would be better. If 1 000 lb. is allowed on the concrete and 16 000 lb. on the steel, the radius of this bend would be, approximately,

$$r = \frac{14\,A}{n} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (\mathrm{IX})$$

Where A = the area of the diagonal and n = the width of the same. For round rods, this would make about

$$r = 10\,d \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (\mathrm{X})$$

Where d = the diameter of the rod.

In this case, however, the stress in the diagonal would be about 40% greater at the lower end of the bend than at its upper end, so

Mr. Jonson. that, instead of Equation VIII, the following formula would express the stress in the 45° diagonal reinforcement in one unit of length:

$$t = \frac{S}{j} \text{..........................} \quad \text{(XI)}$$

Mr. Wason. LEONARD C. WASON, M. AM. SOC. C. E. (by letter).—The writer was considering the submission of a paper intended to draw out a discussion leading to the adoption of a simple formula for general use in designing reinforced concrete beams, when Captain Sewell's paper appeared, therefore this discussion is submitted partly as a discussion of his paper, instead of as an independent paper. Its object is to show that sufficient data are now available to determine an accurate formula (accurate within allowable limits of variation) for general use in designing beams of rectangular or **T**-section, and to emphasize the fact that the best results are obtained by solving for the working instead of the ultimate strength. Therefore this is not exclusively a discussion of Captain Sewell's paper. It is hoped that all those whose formulas are compared herein, and many others, will contribute to the discussion. There are probably at least ten formulas in use besides those mentioned. The writer has discussed the subject chiefly from its commercial aspect, and has purposely omitted the more technical points relating to the disagreement in results, leaving these to be discussed by the professors, who are much better able to do so.

The need of a generally accepted method has been forced upon the writer through competition. In several cases where architects have specified spans and floor loads, and have left the design entirely open to the bidders, work has been lost because competitors have put in lighter designs. In other cases, where a bid has been submitted, accompanied by plans, and, later, the designs have been submitted for review by a consulting engineer, the cost has been increased by more than 10%, due solely to a difference in the formulas. By reference to Table 2, a summary table of comparison, it will be seen that if the work had been designed by the method proposed by the writer, and submitted to an engineer to be checked by the author's formula, the cost would have been appreciably increased, because of the much lower moment of resistance for the same section of beam. Yet structures costing many millions of dollars have been designed by the formula proposed, competition has proved it economical, and the experience of fifteen years has proved it to be safe. There is a general and simple method for designing wooden and steel beams; why should there not be one for reinforced concrete?

The general form of formulas, Equation 0, of Captain Sewell's paper, $M = u\ d\ A\ f$, is correct. The only factor about which there can be much discussion is the value of u.

The formulas which follow—proposed by nine writers—are based on different assumptions, all of which, from certain experiments, appear to have solid foundations. All are based on the same fundamental conditions, namely: Mr. Wason.

1.—All tension is carried by the steel;

2.—All compression is carried by the concrete;

3.—There is a perfect bond or union between the steel and concrete within the limits of the stresses used;

4.—The effects of shear are omitted, and failure is due to flexure only;

5.—There are no initial strains, and the same examples solved by each, using the same constants, ought to give results directly comparable.

The formulas of various writers are reduced to the same general form, and two examples (one a beam and the other a slab) are solved, first, using the same constants for all; secondly, using the constants proposed by each individual writer. The results are summarized in Table 2.

Assume a rectangular beam: span, 14 ft.; width, 12 in.; depth to center of reinforcement, 12 in.; total depth, 13½ in. Find the maximum moment of resistance, load uniformly distributed, area of steel, and neutral axis.

Assume, also, a flat slab: span, 8 ft.; width, 12 in.; depth, 4½ in.; depth to center of reinforcement, 4 in. Find the ultimate moment of resistance, load, area of steel, and position of neutral axis.

Values of Constants, and Definition of Symbols.

Broken-stone concrete. Mixture 1:3:6. Age 30 days.

$E_c = E_t$ =	modulus at working stress	3 000 000
E_s =	modulus of steel	30 000 000
c =	ultimate compression in concrete	2 000
t =	ultimate tension in concrete	200
f =	elastic limit of steel	50 000
f_1 =	working stress in steel	16 000
c_L =	working stress in concrete compression	500

M = moment of resistance of section of beam, in inch-pounds;

W = total uniformly distributed load on beam, in pounds;

E_s = modulus of elasticity of steel;

E_c = " " " " concrete in compression;

E_t = " " " " " " tension;

f = fiber stress in steel, in pounds tension per unit of area;

c = compression in outer fiber of concrete, in pounds per unit of area;

t = tension in outer fiber of concrete, in pounds per unit of area;

Mr. Wason. A = area of steel, in square inches;

p = ratio of area of steel to cross-section of beam.

$n = \frac{E_s}{E_c}$;

b, d, h, x, y, v, e, see Fig. 13;

z and u are ratios of d or h;

l = span of beam, in inches;

V = distance from top of beam to center of gravity of compressive stresses.

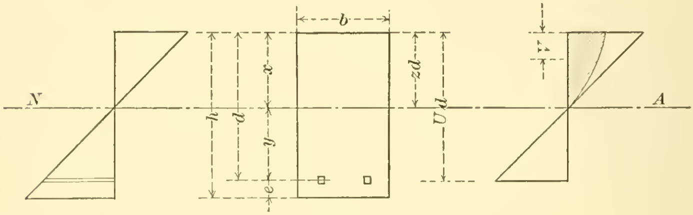


FIG. 13.

1.—The formula proposed by the writer is quite simple, and experience has proved that structures designed by it will carry their calculated load with the desired factor of safety. It assumes the neutral axis half way from the top of the beam to the center of the steel, and the center of gravity of the compressive stress at one-third of the depth from the top to the neutral axis; the compression area is confined to the upper third of the beam. The elastic theory is involved by the values selected for the working stresses of the steel and concrete, and by seeing that the areas of each are sufficient for the stresses used.*

$$A = \frac{b\,d\,c}{3\,f};$$

$$M = \frac{5}{6}\,d\,f\,A:$$

External moment, load uniformly distributed $= \frac{W\,l}{8}$.

Therefore,

$f = \frac{W\,l}{6\frac{2}{3}\,d}$, for a beam supported at its ends and uniformly loaded.

In any given case, the load and span are known. Select a convenient figure for the depth of the beam or the stress in the steel, and solve for the other unknown quantity.

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* For a full demonstration see *Transactions*, Am. Soc. C. E., Vol. XLVI, 1901, p. 102; or *Engineering Record*, September 21st, 1901, p. 272.

Example of Beam: Mr. Wason.

$$A = \frac{b\,d\,c_1}{3\,f_1} = \frac{12 \times 12 \times 500}{3 \times 16\,000} = 1.5 \text{ sq. in.};$$

$$M = \frac{5}{6}\,d\,f\,A = \frac{5}{6} \times 12 \times 50\,000 \times 1.5 = 750\,000;$$

$$W = \frac{750\,000 \times 8}{14 \times 12} = 35\,714.$$

Example of Slab:

$$A = \frac{12 \times 4 \times 500}{3 \times 16\,000} = 0.5;$$

$$M = \frac{5}{6} \times 4 \times 50\,000 \times 0.5 = 83\,333;$$

$$W = 6\,944.$$

There is no change in constants necessary.

2.—W. K. Hatt, Assoc. M. Am. Soc. C. E., has evolved an elaborate formula for flexure.* In the form here given, it applies to rectangular beams which do not fail by shearing. The stresses in the concrete are assumed to follow a parabolic curve. The formula gives the load at the first visible crack in the concrete on the tension side. This load, according to tests by Professor Hatt, is about 20% less than that of ultimate failure.

Let $h\,z$ = the distance from the compression face to the neutral axis;

$h\,u$ = the distance from the compression face to the center of gravity of the reinforcement;

p = the ratio of the area of the steel to that of the total cross-section of the beam;

f = stress at the elastic limit of the steel.

p and u are in the control of the designer; n is fixed for the given materials and working stresses.

After a crack has formed, the neutral axis is located by the formula:

$$\frac{2}{3}\,z^2 = p\,\frac{E_s}{E_c}\,(u - x);$$

$$c = \frac{3\,p\,f}{2\,z};$$

$$M = b\,h^2\left[\frac{5}{12}\,c\,z^2 + p\,f\,(u - z)\right];$$

or, in a simpler form, where the expression in the bracket is represented by the constant, K,

$$M = b\,h^2\,K.$$

* *Engineering News*, February 27th and July 27th, 1902; and *Journal* of the Western Society of Engineers, June, 1904.

Mr. Wason. For given conditions, a table is made for K, in order to simplify the application of the formula; or diagrams of the equation may be made and used. Professor Hatt states that, in his judgment, one-third of the amount at the first crack is the safe working moment of resistance. This would give a factor of safety, on the ultimate strength, of about $3\frac{3}{4}$.

Example of Beam :

$$p = \frac{A}{b\,h} = \frac{1.5}{12 \times 13.5} = 0.0092\,;$$

$$A \text{ (assumed)} = 1.5 \text{ sq. in.}\,;$$

$$u = \frac{12}{13.5} = 0.9\,;$$

$$n = \frac{3\,000\,000}{200 \div \frac{1}{1\,000}} = 15\,;$$

$$\frac{2}{3}\,z^2 = 0.0092\,\frac{30\,000\,000}{3\,000\,000}\,(0.9 - z)\,;$$

$$z = 0.29\,;$$

$$C = \frac{3 \times 0.0092 \times 50\,000}{2 \times 0.29} = 2\,379\,;$$

$$M = 12 \times 13.5^2 \left[\frac{5}{12} \times 2\,379 \times 0.29^2 + 0.0092 \times 50\,000\,(0.9 - 0.29)\right]$$

$$= 795\,959\,;$$

$$W = \frac{795\,959 \times 8}{14 \times 12} = 37\,906\,;$$

$$h\,z = 3.93.$$

Example of Slab :

A (assumed) $= 0.5$; p, u and z, same as above ;

$M = 88\,440$;

$W = 7\,370$.

Professor Hatt's constants :

$E_c = 4\,130\,000$; $n = 16$; $o = 250$.

Then $z = 0.345$, and $c = 2\,000$.

Beam : $M = 775\,226$;

$W = 36\,915$.

Slab : $M = 86\,136$;

$W = 7\,178$.

3.—Edwin Thacher, M. Am. Soc. C. E., proposes the following formula.* In the form here given, it is modified to apply to any width, b, of beam instead of a beam 1 in. wide.

Let f = the stress per square inch on the steel, the gross area = the ultimate strength per square inch of the test piece + 10%;

* *Transactions*, Assoc. of C. E. of Cornell Univ., for 1902.

E_c = the modulus under a pressure of from 1 000 to 2 000 lb. per sq. in.; Mr. Wason.

$$y = \frac{d}{\left(\frac{c}{f} \times \frac{E_s}{E_c} + l\right)};$$

$$x = d - y;$$

$$A = \frac{d\,b}{2\left[\frac{f}{c} + \left(\frac{f}{c}\right)^2 \times \frac{E_c}{E_s}\right]};$$

$$M = \frac{f}{3}\left[\frac{E_c\,x^3}{E_s\,y}\,b + 3\,A\,y\right];$$

$$W = \frac{24\,f}{91}\left[\frac{E_c}{E_s} \times \frac{x^3}{y}\,b + 3\,A\,y\right],$$

for a load uniformly distributed.

To design a beam, assume values of f, E_s, E_c, c, b and d. For other systems of loading or of support, the coefficients in the equations for M and W, outside the bracket, would change.

Example of Beam :

$$y = \frac{12}{\frac{2\,000}{50\,000} \times \frac{30\,000\,000}{3\,000\,000} + 1} = 8.57;$$

$$x = 3.43.$$

$$A = \frac{12 \times 12}{2\left[\frac{50\,000}{2\,000} \times \frac{(50\,000)^2}{2\,000} \times \frac{3\,000\,000}{30\,000\,000}\right]} = 0.82;$$

$$M = \frac{50\,000}{3} \times \left[\frac{3\,000\,000}{30\,000\,000} \times \frac{3.43^3}{8.57} \times 12 + 3 \times 0.82 \times 8.57\right]$$
$$= 445\,499;$$

$W = 21\,214.$

Example of Slab :

$y = 2.86$; $x = 1.14$; $A = 0.274$; $M = 49\,500$; $W = 4\,125$.

Mr. Thacher's units:

Factor of safety, $3\frac{1}{2}$.

$$E_c = 1\,220\,000;\ c = 2\,050;\ f = 64\,000 + 10\%;$$

Beam: $A = \frac{b\,d}{165} = \frac{12 \times 12}{165} = 0.873;$

$$M = 30.62\,d^2 \times 12 \times 12 = 634\,936;$$

$$W = 30\,235;$$

$$y = \frac{12}{\frac{2\,050}{70\,400} \times \frac{30\,000\,000}{1\,220\,000} + 1} = 7.01;$$

$$x = 4.99;$$

Slab: $A = 0.291$; $y = 2.33$; $x = 1.67$; $M = 70\,548$; $W = 5\,879$.

Mr. Wason. *4.*—William H. Burr, M. Am. Soc. C. E., gives these formulas* for rectangular beams for the special case where the tension in the concrete is neglected and the steel is on the tension side only:

$$x = -\frac{E_s\,A}{E_c\,b} \pm \sqrt{\left(\frac{E_s\,A}{E_c\,b}\right)^2 + 2\,\frac{E_s\,A}{E_c\,b}\,d} = \frac{E_s}{E_c} \times \frac{d - x}{x}\,c = 34\ 920;$$

$$M = c\left[\frac{b\,x^2}{3} + \frac{E_s}{E_c} \times \frac{A}{x}\,(d - x)^2\right].$$

Example of Beam:

A (assumed) $= 1.5$;

$$x = -\frac{30\ 000\ 000 \times 1.5}{3\ 000\ 000 \times 12} \pm \sqrt{1.25^2 + 2 \times 1.25 \times 12} = 4.37;$$

$$M = 2\ 000\left[\frac{12 \times 4.37^2}{3} + \frac{30\ 000\ 000 \times 1.5}{3\ 000\ 000 \times 4.37}(12 - 4.37)^2\right] = 552\ 180;$$

$W = 26\ 294.$

Example of Slab:

A (assumed) $= 0.5$;

$$x = -0.417 \pm \sqrt{0.417^2 + 2 \times 0.417 \times 4} = 1.45;$$

$M = 61\ 640;$

$W = 5\ 137.$

Professor Burr's units :

$$c = 3\ 100.$$

Beam : $x = 4.38$; $M = 855\ 879$; $W = 40\ 756.$

Slab : $x = 1.45$; $M = 95\ 480$; $W = 7\ 957.$

5.—A. L. Johnson, M. Am. Soc. C. E., proposes the following :

$f =$ elastic limit of the steel ;

$$y = \frac{2\,f\,E_c}{3\,c\,E_s}\,x; \quad x + y = d\,;$$

$$A = \frac{75\,c\,b\,x}{120\,f}\,;$$

$$M = f\,A\left(y + \frac{2\,x}{3}\right).$$

Example of Beam :

$$y = \frac{2 \times 50\ 000 \times 3\ 000\ 000}{3 \times 2\ 000 \times 30\ 000\ 000}\,x = 1.67\,x\,;$$

$x + y = 12$;

$x = 4.50$;

$y = 7.50$;

$$A = \frac{75 \times 2\ 000 \times 12 \times 4.5}{120 \times 50\ 000} = 1.35\,;$$

* "The Elasticity and Resistance of the Materials of Engineering," 1903 edition.

Mr. Wason.

$$M = 50\,000 \times 1.35 \left(7.50 + \frac{2 \times 4.5}{3}\right) = 708\,750\,;$$

$$W = 33\,750.$$

Example of Slab :

$$y = 2.5\,; \quad x = 1.5\,; \quad A = 0.45\,;$$

$$M = 78\,750\,; \quad W = 6\,563.$$

Mr. Johnson's units :

$$E_s = 29\,000\,000\,; \quad y = 1.72\,x\,; \quad x = 4.41\,; \quad A = 0.0195\,b\,x = 1.32\,;$$

$$M = 2\,750\,b\,x^2 = 641\,850\,; \quad W = 30\,564.$$

Slab : $x = 1.47\,; \quad A = 0.34\,; \quad M = 71\,280\,; \quad W = 5\,940.$

6.—J. Kahn, Assoc. M. Am. Soc., C. E., uses a compressive area different from that used by any other writer. See Fig. 14.

$f =$ the ultimate tensile stress in the steel ;

$$y = \frac{15\,A + b\,d^2}{30\,A + 2\,b\,d}\,;$$

$$M = f\,A\,\left(\frac{5}{8}\,x + y\right)\,;$$

b should never be less than $\dfrac{A\,f}{1\,800\,x}$.

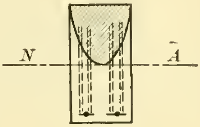

FIG. 14.

Example of Beam :

$$y = \frac{15 \times 1.5 + 12 \times 12^2}{30 \times 1.5 + 2 \times 12 \times 12} = 5.26\,;$$

A (assumed) $= 1.5$;

$$M = 50\,000 \times 1.5\,\left(\frac{5}{8} \times 6.74 + 5.26\right) = 710\,250;$$

$$W = 33\,821.$$

Example of Slab :

A (assumed) $= 0.5\,; \quad y = 1.80\,; \quad M = 79\,375\,; \quad W = 6\,614.$

Mr. Kahn's units :

$$E_c = 2\,000\,000\,; \quad c = 3\,000\,; \quad f = 64\,000\,;$$

A (assumed) $= 1.5\,; \quad y = 5.26\,;$

$$M = 64\,000 \times 1.5\left(\frac{5}{8} \times 6.74 + 5.26\right) = 909\,120\,;$$

$$W = 43\,291.$$

Slab : $y = 1.80\,; \quad M = 101\,600\,; \quad W = 8\,467.$

7.—A. N. Talbot, M. Am. Soc. C. E., proposes the following,* which is the special case for the ultimate deformation of concrete :

Let $p = \dfrac{A}{b\,d} =$ ratio of area of metal reinforcement to area of concrete above center of reinforcement ;

* *Engineering Record*, August 13th, 1904; and *Journal* of Western Society of Engineers, August, 1904.

Mr. Wason.

$$z = \sqrt{\frac{2\,p\,n}{1 - \frac{1}{3}} + \frac{p^2\,n^2}{(1 - \frac{1}{3})^2}} - \frac{p\,n}{1 - \frac{1}{3}};$$

$$V = \frac{3}{8}\,z\,d;$$

$$M = A\,f\,(d - V).$$

The solution of the values of z and V for smaller stresses than the ultimate is somewhat complicated. However, using the values of z obtained by tests of 1:3:6 concrete beams, the formula becomes:

$$M = A\,f\,(0.906 - 6.5\,p)\,d.$$

This is a simple and convenient form to use.

Example of Beam:

A (assumed) $= 1.5$;

$$p = \frac{1.5}{12 \times 12} = 0.01;$$

$$n = \frac{30\,000\,000}{3\,000\,000} = 10;$$

$$z = \sqrt{\frac{2 \times 0.01 \times 10}{\frac{2}{3}} + \frac{0.01^2 \times 10^2}{\frac{2}{3}}} - \frac{0.01 \times 10}{\frac{2}{3}} = 0.418;$$

$$V = \frac{3}{8} \times 0.418 \times 12 = 1.88;$$

$$M = 1.5 \times 50\,000 \times (12 - 1.88) = 759\,000;$$

$$W = 36\,143.$$

Example of Slab:

$$V = 0.63;$$

$$M = 84\,325;$$

$$W = 7\,027.$$

Professor Talbot's units:

E_c = initial modulus average, 2 000 000; $n = 15$; $z = 0.482$; $V = 2.17$; $M = 1.5 \times 50\,000 \times 9.83 = 737\,250$; $W = 35\,107$.

Slab: $V = 0.72$; $M = 82\,000$; $W = 6\,833$.

8.—John S. Sewell, M. Am. Soc. C. E.

$$x + y = d;$$

$$y = \frac{f_1\,E_c}{c_1\,E_s}\,x;$$

$$y = \frac{f\,E_c}{0.8\,c\,E_s}\,x;$$

$$u = 0.64 + y.$$

Let $c_1 = 0.8\,c$;

$$x - V = 0.64\,x;$$

area under curve $= 0.57\,c_1\,x = 0.456\,c\,x$;

$$0.465\,b\,c\,x = A\,f;$$

$$M = 0.456 \times 0.64\,b\,c\,x^2 + A\,f\,y = 0.292\,b\,c\,x^2 + A\,f\,y.$$

Mr. Wason.

Example of Beam:

$$y = \frac{50\ 000 \times 3\ 000\ 000}{0.8 \times 2\ 000 \times 30\ 000\ 000}\, x = 3.125\, x;$$

$x = 2.91$; $y = 9.09$; $u = 10.95$;

$$A = \frac{0.456 \times 12 \times 2\ 000 \times 2.91}{50\ 000} = 0.637;$$

$M = 0.292 \times 12 \times 2\ 000 \times 2.91^2 + 0.637 \times 50\ 0000 \times 9.09 = 348\ 860$;
$W = 16\ 612$.

Example of Slab:

$x = 0.97$; $y = 3.03$; $A = 0.212$; $u = 3.65$;
$M = 0.292 \times 12 \times 2\ 000 \times 0.97^2 + 0.212 \times 50\ 000 \times 3.03 = 38\ 706$;
$W = 3\ 225$.

Captain Sewell's units:

$$f = 45\ 000; \quad c = 2\ 500; \quad \frac{E_c}{E_s} = \frac{1}{15}.$$

Beam:

$$y = \frac{45\ 000 \times 1}{0.8 \times 2\ 500 \times 15}\, x = 1.5\, x; \quad x = 4.8; \quad y = 7.2; \quad u = 10.27;$$

$$A = \frac{0.456 \times 12 \times 2\ 500 \times 4.8}{45\ 000} = 1.46;$$

$M = 0.292 \times 12 \times 2\ 500 \times 4.8^2 + 1.46 \times 45\ 000 \times 7.2 = 674\ 870$;
$W = 32\ 137$.

Slab:

$x = 1.6$; $y = 2.4$; $A = 0.486$; $u = 3.42$;
$M = 0.292 \times 12 \times 2\ 500 \times 1.6^2 + 0.486 \times 45\ 000 \times 2.4 = 74\ 914$;
$W = 6\ 243$.

9.—Mr. F. D. Warren has written a handbook * on reinforced concrete, in which there are a great many tables. In order to determine their value, the formulas based on the method of the late J. B. Johnson, M. Am. Soc. C. E., published in *Engineering News* in 1895, is submitted for review. An area of concrete equal to ten times the area of the steel is put in the same plane. The moment of inertia of this section is found, and the location of the neutral axis.

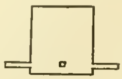

Fig. 15.

$$\frac{E_s}{E_c} = 10;$$

M_o = working moment, factor of safety, $3\frac{1}{2}$;

$$\frac{M_o}{500} = \frac{1}{4}\frac{b\, d^3}{d}$$ Preliminary step to find stress in compression in concrete; with factor of safety of 3.5 and c = 3 000.

* "A Handbook on Reinforced Concrete," pp. 78-81.

Mr. Wason.

$$\frac{M_o}{f_1} = \frac{A\ h^2}{h}$$ $\left\{\right.$ Preliminary step to find area of steel; neutral axis assumed to be from 1.5 to 2.0 in. below the center of gravity of the section.

Transpose this area of steel into an area of concrete, and solve for the moment of inertia and the position of the neutral axis, neglecting the area of the concrete below the plane of the steel. If the neutral axis differs materially from 1.5 to 2.0 below the center of gravity, use this value to determine h in the formula, $\frac{M_o}{f_1} = A\ h$, and with the new value again solve for A. Then check the fiber stress of the concrete in compression.

Example of Beam:

$$\frac{M_o}{333} = \frac{1}{4}\frac{b\ d^3}{d}, \text{ when } c = 2\,000;$$

$M_o = 83.33\ b\ d^2 = 83.33 \times 12 \times 12^2 = 144\,000$, working load;

$M = 144\,000 \times 3.5 = 504\,000$, ultimate load;

$$\frac{M}{f} = \frac{A\ y^2}{y};$$

y (assumed) $= 4.5$;

$$A = \frac{M}{f\,y} = \frac{504\,000}{50\,000 \times 4.5} = 2.24.$$

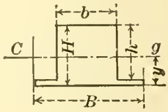

FIG. 16.

Assume 1-in. bars.

$$y = [B\,H - h\,(B - b)] = \frac{1}{2}\ B\ H^2 - h\ (B - b)\ (H - \frac{h}{2});$$

$$y = [34.4 \times 12.5 - 11.5\ (34.4 - 12)]$$

$$= 0.5 \times 34.4 \times 12.5^2 - 11.5\ (34.4 - 12)\ (12.5 - \frac{11.5}{2});$$

$$y = \frac{748.7}{172.4} = 4.34\,;$$

$x = 8.16$, or 2.16 below the central axis;

$$I = \frac{12 \times 12^3}{12} + 12 \times 12 \times 2.16^2 = 2\,400;$$

$$\frac{I}{x} = \frac{2\,400}{8.16} = 294;$$

$$c = \frac{504\,000}{294} = 1\,714;$$

$$A = \frac{M}{f\ y} = \frac{504\,000}{50\,000 \times 4.34} = 2.32.$$

Example of Slab:

$M = 83.33 \times 3.5 \times 12 \times 4^2 = 56\,000$;

Mr. Wason.

$$A = \frac{56\,000}{50\,000 \times 1.5} = 0.75;$$

y (assumed) $= 1.5$.

Assume $\frac{1}{2}$-in. bars.

$$y = 19.5 \times 4.25 - 3.75\,(19.5 - 12)$$
$$= 0.5 \times 19.5 \times 4.25^2 - 3.75\,(19.5 - 12)\left(4.25 - \frac{3.75}{2}\right);$$

$$y = \frac{109.7}{54.88} = 2.00;$$

$x = 2.25$, or 0.25 below the central axis:

$$I = \frac{12 \times 4^3}{12} + 12 \times 4 \times 0.25^2 = 68.0;$$

$$\frac{I}{x} = 30.7;$$

$$A = \frac{56\,000}{50\,000 \times 2.0} = 0.56.$$

F. D. Warren's constants:

Beam: $c = 3\,000$; $f = 53\,000$; $f_1 = 15\,000$.

Factor of safety $= 3.5$.

$M_o = 125\,b\,d^2$;

$M = 125\,b\,d^2 \times 3.5 = 125 \times 12^2 \times 12 \times 3.5 = 756\,000$;

$A = \frac{M}{f\,y} = \frac{756\,000}{53\,000 \times 4} = 3.57$; y (assumed) $= 4$; 1-in. bars assumed.

$$y = 47.7 \times 12.5 - 11.5\,(47.7 - 12)$$
$$= 0.5 \times 47.7 \times 12.5^2 - 11.5\,(47.7 - 12)\left(12.5 - \frac{11.5}{2}\right);$$

$$y = \frac{955.3}{185.7} = 5.14;$$

$x = 7.36$, or 1.36 below the central axis;

$$I = \frac{12 \times 12^3}{12} \div 12 \times 12 \times 1.36^2 = 1\,994.6;$$

$$\frac{I}{x} = 271;$$

$$c = \frac{756\,000}{271} = 2\,790;$$

$$A = \frac{756\,000}{53\,000 \times 5.14} = 2.78.$$

Slab:

$M = 125 \times 12 \times 4^2 \times 3.5 = 84\,000$;

$$A = \frac{84\,000}{53\,000 \times 1.75} = 0.906;$$

y (assumed) $= 1.75$;

Mr. Wason.

$$y = [21 \times 4.25 - 3.75\,(21 - 12)]$$

$$= 0.5 \times 21 \times 4.25^2 - 3.75\,(21 - 12)\left(4.25 - \frac{3.75}{2}\right);$$

$$y = \frac{109.6}{55.5} = 1.98\,;$$

$x = 2.27$, or 0.27 below the central axis;

$$A = \frac{84\,000}{53\,000 \times 19.8} = 0.80$$

An examination of the results in Table 2 will show that in three of the nine there is a fairly close agreement with each other, whether with the use of the same constants for all, or the constants recommended by each author, although the assumptions of each are quite different. The three which agree fairly well are: Wason, Hatt, and Talbot. These three, then, would appear to be the most general in character, and the writer's method gives the safest result with the same constants. Professor Talbot's analysis appears to be the most rational solution of the problem, from a profoundly scientific standpoint.

Without doubt, the stress diagram of the concrete in compression is a curve somewhat resembling a parabola. No allowance for tension of concrete should be made. There seems to be, among many writers, a lack of appreciation of the fact that results obtained by solving for the ultimate strength of a reinforced concrete beam, then dividing by a given factor of safety, or using the corresponding working stresses to solve for the working strength of a beam, do not give the same results or results which can be compared. To illustrate: Take the beam previously considered, and the ultimate strength of the concrete as the force, instead of the area of the steel, with the parabolic treatment of the compression, and the neutral axis at one-half the depth to the center of the reinforcement.

Then, arm of ultimate moment $= \frac{5}{8} \times \frac{1}{2}\,d + \frac{1}{2}\,d = 0.8125\,d\,;$

compressive force of concrete $= \frac{5}{8}\,c \times \frac{1}{2}\,d\,b = 0.3125\,c\,d\,b;$

$$M = 0.254\,c\,d^2\,b = 0.254 \times 2\,000 \times 12 \times 12^2 = 877\,824;$$

$\frac{1}{4}\,M =$ working moment;

$M_o = 219\,456.$

If we use an outside fiber stress of one-quarter of the ultimate, namely, $c^1 = 500$, and solve for the working moment of resistance with the triangular, straight-line treatment of compression (see Fig. 17), then,

Arm of working moment $= \frac{2}{3} \times \frac{1}{2}\,d + \frac{1}{2}\,d = 0.833\,d;$

compressive force of concrete $= \frac{1}{2}\, c^1 \times \frac{1}{2}\, b\, d = 0.25\, c^1\, b\, d$; Mr. Wason.

$M_0 = 0.208\, c^1\, b\, d^2 = 180\,000.$

The correct method is to solve for the working strength. Assuming a working compressive stress of one-fourth the ultimate, the difference between the straight line and the parabola is negligible, and the triangular area can be used, as it is simpler. The variation in the quality of the best concrete will more than offset the refinement of retaining the curve. If the triangular area is used for ultimate compression, the result will be too large, in the ratio of $\frac{1}{2}$ to $\frac{5}{16}$, or $37\frac{1}{2}$ per cent. The modulus of elasticity is used only in finding the position of the neutral axis, in order to find the center of gravity of the compressive force. The position of the neutral axis is of very little value.

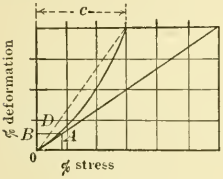

FIG. 17.

FIG. 18.

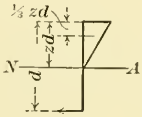

FIG. 19.

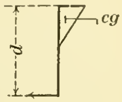

FIG. 20.

To illustrate, take two extreme cases: First, $E_c = 4\,500\,000$ and $\frac{1}{2}$% of steel; secondly, $E_c = 1\,250\,000$ and 2% of steel. Then, by Talbot's formula: First case, $z = 0.266$; second case, $z = 0.616$.

$$\text{Arm of moment} = d - \frac{1}{3}\, z\, d$$

First case	"	"	"	$= 0.925\, d$
Second case	"	"	"	$= 0.795\, d$
Mean............				$= 0.86\, d$

The difference is 0.13 d or 14 per cent. If a factor of safety of 4 is used on the crushing strength of concrete, then the 14% of ultimate strength is only $3\frac{1}{2}$% of the working strength. Greater differences are frequently found among several test specimens made from the same batch. As the case cited covers a wide range, from a rich mixture at an age of several months, with insufficient reinforcement, to a lean mixture at the age of one month, with an excess of metal, with an error negligible in common practice, it is evident that no change is necessary in the formula for any ordinary cases of construction, because they always fall within these limits. This adds greatly to the simplicity of making designs, and to universality in the use of tables.

Mr. Wason.

TABLE 2.—SUMMARY OF RESULTS USING THE SAME CONSTANTS:

FOR BEAMS.

Author.	Ultimate bending moment, in pounds.	Breaking load, in pounds.	Area of steel, in square inches.	Stress in steel, in pounds per square inch.	Distance, top to neutral axis, in inches.	Arm of moment of resistance, in inches.	Arm, as percentage of *d*.
Wason......	750 000	35 714	1.5	50 000	6.00	10.00	0.833
Hatt.........	795 939	37 906	1.5	50 000	3.93	10.53	9.877
Thacher....	445 499	21 214	0.82	50 000	3.43	10.86	0.905
Burr.........	552 180	26 294	1.50	34 920	4.37	10.54	0.878
Johnson....	708 750	33 750	1.35	50 000	4.50	10.32	0.86
Kahn........	710 250	33 821	1.50	50 000	6.74	9.47	0.789
Talbot......	759 000	36 143	1.50	50 000	5.02	10.12	0.843
Sewell......	348 860	16 612	0.64	50 000	2.91	10.95	0.913
Warren.....	504 000	24 000	2.24	50 000	8.16	9.28	0.773

FOR SLABS.

Author.	Ultimate bending moment, in pounds.	Breaking load, in pounds.	Area of steel, in square inches.	Stress in steel, in pounds per square inch.	Distance, top to neutral axis, in inches.	Arm of moment of resistance, in inches.	Arm, as percentage of *d*.
Wason......	83 333	6 944	0.5	50 000	2.00	3.33	0.833
Hatt.........	88 440	7 370	0.5	50 000	1.31	3.51	0.877
Thacher....	49 500	4 125	0.28	50 000	1.14	3.62	0 905
Burr.........	61 640	5 137	0.5	34 920	1.45	3.52	0.878
Johnson....	78 750	6 563	0.45	50 000	1.50	3.44	0.86
Kahn........	79 375	6 614	0.5	50 000	2.20	3.18	0.795
Talbot......	84 325	7 027	0.5	50 000	1.67	3.37	0.842
Sewell......	38 706	3 225	0.21	50 000	0.97	3.65	0.913
Warren....	56 000	4 667	0.57	50 000	2.25	3.25	0.812

SUMMARY OF RESULTS USING EACH AUTHOR'S CONSTANTS:

FOR BEAMS.

Author.	Ultimate bending moment, in pounds.	Breaking load, in pounds.	Area of steel, in square inches.	Stress in steel, in pounds per square inch.	Distance, top to neutral axis, in inches.	Arm of moment of resistance, in inches.	Arm, as percentage of *d*.
Wason......	750 000	35 714	1.5	50 000	6.00	10.00	0.833
Hatt.........	775 226	36 915	1.5	50 000	4.66	10.25	0.854
Thacher....	634 936	30 235	0.87	70 400	4.99	10.33	0.861
Burr........	855 879	40 756	1.5	53 940	4.38	10.54	0.878
Johnson....	641 850	30 564	1.32	50 000	4.41	10.35	0.863
Kahn........	909 120	43 291	1.5	64 000	6.74	9.47	0.789
Talbot......	737 250	35 107	1.5	50 000	5.78	9.83	0.819
Sewell......	674 870	32 137	1.46	45 000	4.8	10.27	0.856
Warren....	756 000	36 000	2.78	53 000	7.36	9.55	0.792

FOR SLABS.

Author.	Ultimate bending moment, in pounds.	Breaking load, in pounds.	Area of steel, in square inches.	Stress in steel, in pounds per square inch.	Distance, top to neutral axis, in inches.	Arm of moment of resistance, in inches.	Arm, as percentage of *d*.
Wason......	83 333	6 944	0.5	50 000	2.00	3.33	0.833
Hatt........	86 136	7 178	0.5	50 000	1.55	3.42	0.855
Thacher....	70 548	5 875	0.29	70 400	1.67	3.44	0.86
Burr.........	81 933	6 828	0.5	53 940	2.15	3.28	0.82
Johnson....	71 280	5 940	0.34	50 000	1.47	3.45	0.862
Kahn........	101 600	8 467	0.5	64 000	2.20	3.18	0.795
Talbot......	82 000	6 833	0.5	50 000	1.93	3.28	0.82
Sewell......	74 914	6 243	0.49	45 000	1.6	3.42	0.855
Warren.....	84 000	7 000	0.80	53 000	2.27	3.24	0.810

In a large number of tests examined, the neutral axis for one-quarter of the ultimate load has been found to be not far from one-half the depth from the top of the beam to the center of reinforcement. It is sometimes above and sometimes below. To assume it half way, agrees very closely, therefore, with the observed facts. With this position and a straight-line distribution within the working stress, the center of gravity of the compressive stresses is two-thirds of the distance up from the neutral axis to the top of the beam. Therefore, the arm of the moment of resistance is $\frac{5}{6}\, d = 0.833\, d$. Mr. Wason.

The method of designing beams with a **T**-section need not add complications. The foregoing formula can be applied without difficulty. The same value of u can be used as for a rectangular beam, inasmuch as the center of gravity of the compressive forces for the **T**-section, usually, will nearly coincide with that for the rectangular one, and if it does not, u will be less than its true value, therefore the beam will be somewhat heavier than is necessary. If the depth and size of the steel are fixed, it is then merely necessary to find a sufficient area of concrete to balance this; or take the maximum allowable width of flange, which usually may be taken as four times the width of the web, and from the other known dimensions of the section, the area in compression may be obtained. Knowing the ratio of the allowable working stresses in the concrete and the steel, the area of the steel is fixed. One advantage of this formula is that the area of the concrete above the neutral axis multiplied by its average stress may be substituted for the area of the steel.

In the writer's practice, he has never known a case where a floor was composed of concrete leaner than 1 : 3 : 6, or richer than 1 : 2 : 4, and as a designer will generally use one mixture at all times, there is no complication by a large number of ratios of areas of steel to concrete, as the percentage is a fixed amount determined entirely by the working stress of each material. In using such a formula, it is as easy to check existing work designed by others as to design new work independently by it.

In using the method proposed by Professor Hatt, there are too many variables depending upon one another. The position of the neutral axis varies with the percentage of the area of reinforcement to that of the concrete; the ratio of moduli of elasticity of the two materials, which in turn varies with the mixtures, age, and character of the cement, sand and stone; and the stress-strain curve differs for each case. Moreover, the position of the neutral axis is of little consequence. Its position is not accurately known at the ultimate strength of the beam, therefore the value of c, corresponding with a given f, is only approximate. The location of the first crack is also too variable to be a safe guide to the proper working load. He places the steel too near the bottom of the beam.

Mr. Wason. The method proposed by Mr. Thacher appears to be in error in finding the value of A, which is carried into the solution of the moment of resistance. The writer has never found the percentage of steel per inch of width of beam useful in designing structures.

Professor Burr places the ultimate strength of the concrete too high, and does not get full efficiency from the steel.

Mr. A. L. Johnson stakes his reputation and his design on the fact that reinforced concrete stretches more than plain concrete; to quote his own words:*

"As stated in the introduction, this effect of the embedded metal upon the extensibility of the concrete is the quality upon which the whole art of steel-concrete construction rests. Fortunately, it is a quality as to the existence of which there can be no doubt. The art had been a success for many years before science stepped in to tell us why."

Mr. Kahn's method the writer has been unable to check by any of the ordinary principles of design, and he sees no logical reason for the area used in compression.

Professor Talbot's formula is the most rational of any yet proposed, covering every point, and it can be readily applied to any degree of loading. It is in the form proposed for the universal formula.

Captain Sewell bases his determination for the position of the neutral axis upon the ratio of the moduli of elasticity and the working stress of the two materials. As these do not vary proportionally, results based upon this method are inaccurate. Moreover, with concrete wherein the modulus and stress are not proportional, the assumption that a plane before bending is a plane after bending is open to doubt. He states that the values of c and E_c must always be those that correspond to each other. The value of c differs somewhat with different brands of cement, and, as the designer does not know what brand will be used in his work, the correct ratio of c to E_c may not be used, and a safe value for all brands may not be economical. The writer's study of recent tests at the Watertown Arsenal led chiefly to the conclusion that it is best not to use a certain brand which was largely used in making these tests. The ultimate strength of several brands varied widely, yet the modulus at the same stress, approaching the ultimate load, varied but little. This is somewhat similar to the variation in the elastic limit and ultimate strength of steel with a change in the percentage of carbon, while the modulus of elasticity remains constant. This is a serious defect in the formulas involving ratios of these stresses.

Mr. Warren's method is a series of approximations, and the result is the same. Moreover, using his method involves a longer task

* Catalogue of St. Louis Expanded Metal Fireproofing Company, 1903, p. 53.

than any of the others. The result shows an extravagant use of steel and a lack of efficiency in the concrete. The stress in this material is not as great as his formula would indicate. A thickness of 1 in. of concrete below the steel is not enough when the size of the bar exceeds $\frac{3}{4}$ in. square. Mr. Wason.

Some engineers solve for the working stress of the beam, and use the parabolic curve for the compressive stresses. This is inaccurate, inasmuch as, within the working stresses, the stress is almost exactly proportional to the strain, so that a plot is either a straight line or its difference from a straight line is negligible. With this method of design, the outside fiber stress in compression is actually about $1\frac{1}{2}$ times as great as it is assumed, as shown below.

For a parabolic curve: $c = 500$; $p = 0.01$; $n = 10$; $x = 0.42$.

$$\frac{M}{b\,d^2} = \frac{2}{3}\,c\,x\left(1 - \frac{3}{8}\,x\right) = \frac{2}{3} \times 500 \times 0.42\left(1 - \frac{3}{8} \times 0.42\right)$$

$$= 140\,x \times 0.843 = 118.$$

For a straight line: $c = 750$; $p = 0.01$; $n = 10$; $x = 0.36$.

$$\frac{M}{b\,d^2} = \frac{c}{2}\,x \times \left(1 - \frac{x}{3}\right) = \frac{1}{2} \times 750 \times 0.36\left(1 - \frac{0.36}{3}\right)$$

$$= 135 \times 0.88 = 118.8.$$

x is taken from diagrams, using p as ordinates and x as abscissas for given values of n, x being deduced from the formula for a parabola:

$$x = -\frac{3}{2}\,n\,p + \sqrt{\left(\frac{3}{2}\,n\,p\right)^2 + 3\,n\,p};$$

and for a straight line:

$$x = -\,n\,p + \sqrt{(n\,p)^2 + 2\,n\,p}.$$

These plots were made by Professor Arthur W. French. It will be seen that the results are almost identical, although $c = 500$ in one case and 750 in the other.

Captain Sewell's taking 80% of the ultimate strength as a maximum stress on the concrete when the steel reaches its elastic limit is apparently based on Professor Hatt's tests at the first crack. The working stress obtained in concrete on this basis, using $2\frac{1}{2}$ as a factor of safety, or 32% of the ultimate strength, seems to be reasonable. The writer would recommend using the sum of the dead and live loads as the load on which the factor is based, except in the rare cases where the dead load is larger than the live load, when (live load $+ \frac{4}{10}$ dead load) $\times 2\frac{1}{2}$ = the elastic limit of the steel, may be used. The limit of elastic deformation determined by Professor Bauschinger, which the writer has never seen questioned, was about seven-tenths of the ultimate resistance; therefore, approxi-

Mr. Wason. mately, the above factor of safety is based on the elastic limit of the concrete as well as of the steel.

In designing, it is assumed that there are no initial strains in the composite structure, but, nevertheless, they exist, as the shrinkage of the cement in setting produces an initial compression in the steel which the formula does not recognize, and this, if neglected, adds to the factor of safety. Inasmuch as the amount of stress cannot be accurately determined, this is the wiser course.

Captain Sewell states that beams designed by the right line are heavier than necessary, and that the parabola rather inclines to the opposite extreme. When solving for the working stresses, this is of less consequence. His value, $u = 0.85$, agrees quite closely with the mean of the two extreme cases previously solved by Professor Talb t's formula, and is sufficiently accurate to be generally used if th ved stress-strain principle be accepted as correct. The writer believes it is not correct when working stresses are considered.

The elaborate discussion as to the most economical design, considering the cost of the steel and the concrete, is very interesting and suggestive, but, inasmuch as the cost of the two materials is entirely independent of their ratio of stresses in practical application, the writer has found that it is sufficiently accurate and economical to design beams with the maximum allowable depth, in order to obtain the maximum of economy.

From every test which has come to the writer's attention, he is convinced that there is no economy in reinforcing the beam with steel against compression, and is glad to see this opinion confirmed. There is not as great economy in using it in compression as in tension, but the writer would go still further and say, from his present knowledge, that it is never advisable to use steel in compression in beams. The tests made for him at the Massachusetts Institute of Technology showed no value whatever in top reinforcement, and to rely on it would be dangerous.

There are other reasons to prevent a theoretically accurate design from being actually carried out on the work. As the ultimate strength and modulus of elasticity increase with age, while the steel does not, the design of the beam becomes unbalanced, thus changing the ratio of the moduli of the two materials, the position of the neutral axis, and, especially important, the ultimate compressive stress, thus changing the expected result from any design where these items are used as factors. Actual transverse tests, where the neutral axis has been located, show that the outside fibers sustain a greater stress than in columns uniformly compressed.

On work of any size, the concrete is mixed by machinery, while laboratory tests are almost invariably mixed by hand. Samples of hand-mixed concrete from actual work give better results than labo-

Mr. Wason.

ratory specimens. In two sets of tests, reported at the Watertown Arsenal in 1897 and 1900, the difference in strength between hand-mixed and machine-mixed concrete, where both cases were exceptionally well done, was 11% in favor of the machine-mixed. In the other set, under commercial conditions, the difference was 25% in favor of the machine-mixed. There is quite a difference in the modulus of machine-mixed and hand-mixed concrete. Wet or plastic mixtures thoroughly or lightly tamped affect the density and the bond with steel.

The bond or union between the steel and the concrete increases with age. The adhesion does not reach a very high value in one month, but increases steadily beyond this point for a number of months. Thus, a design based on adhesion which would not be safe at one month would be safe at a greater period. A mechanical bond is always to be preferred to simple adhesion, however. While the formulas given take no notice of this, the designer must, before completing his work.

Fortunately, all these variations are in the direction of safety, but when natural causes, of which the most elaborate formula can take no cognizance, produce such differences, why attempt great refinement, especially when the values of the several factors cannot be determined with absolute precision?

The discussion resolves itself into:

1. Should the ultimate or the working stresses be used;

2. What arm should be given to the internal moment of resistance, the formula to be used being in the form:

$$M = u\ d\ A\ f.$$

The arm proposed by the writer, $\frac{5}{6}\ d$, or 0.833 d, is 1.7% safer than that proposed by Captain Sewell, and he believes it is more nearly correct.

There are many rules in practice for the quantity of concrete necessary to embed the steel in the bottom of a beam. The practice followed by the writer, which has proved entirely satisfactory in twelve years of practice, is to use a quantity equal to twice the diameter of the bar below its center. In work of exceptional importance, or where there is unusual danger from exposure to fire, this may be increased to $2\frac{1}{2}$ times the diameter of the bar, and this will be found to be ample in the worst cases. The width of the beam should be at least equal to 3 times the diameter of the bar. If this is used as a minimum, there is danger in some cases of shear cracks. Adding an inch to the amount above given, or, at most, 4 times the section of the bar, will be found sufficient for beams reinforced with a single bar. Where more than one is used, allow a clear space between the bars equal to their diameter.

Mr. Wason. A good deal has been said by various writers about the percentage of steel to use. The limiting factor is the area of the concrete. After once determining the working stress of this, the percentage of steel is fixed for any given stress, and this ratio is a constant.

Among the advantages of solving for the working stress, in addition to those previously advanced, is the fact that a great many tests of the crushing strength of concrete of different mixtures and brands of cement have been made when measurements have not been taken to determine its elastic quality. A general average of all these is easy to obtain, or an average omitting the exceptionally high ones, and this may be used as the ultimate strength. In obtaining results of this character, there is no danger of misinterpreting their meaning. The variations in the actual results from those theoretically expected are well within the limits of allowable error. The advantage in commercial practice of all work of different designers agreeing in their principle with one another is as important as in designs made for wood and steel.

In all the foregoing, nothing has been said about web stresses. When the writer began constructing this class of work, twelve years ago, he did not know enough to reinforce the beams against shear. There are a good many structures without any reinforcement which have proved entirely satisfactory in actual service. On account of this experience, he still continues to build many floors for light loads without any web reinforcement. There is one building, however, in which the floors are habitually heavily overloaded by the shock of falling weights. In this, some beams have been cracked, which would indicate there is not a very large factor of safety, and therefore reinforcing against web stresses is desirable. From the experience of actual structures without web reinforcement, and from laboratory tests, it is evident that the concrete will withstand a considerable part of the web stresses, enabling the beam to develop nearly the ultimate strength of its tension and compression members. The writer dissents, therefore, from Captain Sewell's opinion that concrete should no more be relied upon for connections than for resisting tensile flange stresses. Especially is this true when economy is considered.

When beams are to carry a quiet load, or one with but little vibration, the concrete can be allowed to resist shear to the amount of 80 lb. per sq. in. But when there is heavy vibration or sudden shocks, reinforce for the entire web stress. On account of the frequent lapse of time between filling a beam and spreading the panel or flange portion of the floor, it is advisable to use stirrups bent to extend at least 1 ft. each way from the beam into the panel to bond the two together. This is seldom done. It is most needed at the center of the span. Deformed bars are far superior to plain ones, and, now

that there is little difference in price between them, there is no excuse whatever for the use of plain bars. Mr. Wason.

In comparative tests, where some of the main bars were bent diagonally upward, reaching the top of the beam near the support, the greatest strength was obtained—25% greater than with the use of diagonal stirrups attached rigidly to the main bars. This latter method, however, is far superior to diagonal stirrups which are loose, as in tests of this kind, when the beam deflected, they dragged under the main bars, breaking away the concrete and causing failure at a lower load than a beam which had no web reinforcement whatever. Loose vertical stirrups gave better results than beams without any web members. It is harder work to embed diagonal web members properly, as shown on page 657,* than with vertical ones, or where some of the main bars are bent diagonally upward to the top of the beam. Therefore, to obtain as good results with the inclined stirrups requires more expense and smaller stone in the concrete than is required with other types.

This discussion, however, can be made entirely independent of that of flexure, and final judgment should be reserved until more experimental data have been obtained. Some of the colleges are conducting such experiments at the present time, the results of which it is expected will throw considerable light upon this subject. The writer, therefore, has merely stated his experience, which may help toward the ultimate solution.

E. P. Goodrich, M. Am. Soc. C. E.—For several years the speaker has had in mind the use of a simple formula, such as the one suggested by Captain Sewell, to be used in the design of reinforced concrete beams. With this in view, the results of tests of several hundred beams have been carefully analyzed and plotted. After considerable work had been done, the speaker's attention was called to a paper by T. L. Condron, M. Am. Soc. C. E., entitled "A Study of Tests of Reinforced Concrete Beams," presented before the American Society for Testing Materials. In it Mr. Condron has analyzed eighty-three tests, and has proposed a formula of the form. Mr. Goodrich.

(Moment of forces) $\div$ (depth $\times$ area of beams) $=$ (constant $\times$ percentage of steel) $+$ (another constant).

The paper is most interesting and is worthy of close study.

Several engineers had been making use of simplified formulas before L. J. Johnson, M. Am. Soc. C. E., published, in *Engineering News*, his table of the values of K (which corresponds with Captain Sewell's h). As far as the writer is aware, however, Professor Johnson was the first to put the theoretical side of the subject into such shape that a simple formula was available for practical use.

Because of the diversity of composition and age of the tested

* *Proceedings*, Am. Soc. C. E., for December, 1905.

Mr. Goodrich. beams, reports of which were available for examination by the speaker, it was difficult to determine the accuracy of Mr. Condron's deductions. The speaker, therefore, instituted the following test: Seven beams were built, with tension rods only, and all had the same area of steel and the same area of concrete above the reinforcement. The breadth and depth of the several beams were varied as far as possible. With these conditions, the breaking load should vary directly with the depth of the beam, according to Mr. Condron's equations. The beams broke with remarkable uniformity, all failing by crushing.

Fig. 21 shows the plotted results. The results also clearly show the possibility of using such a formula as that proposed by the author, at least when a single percentage of steel is used.

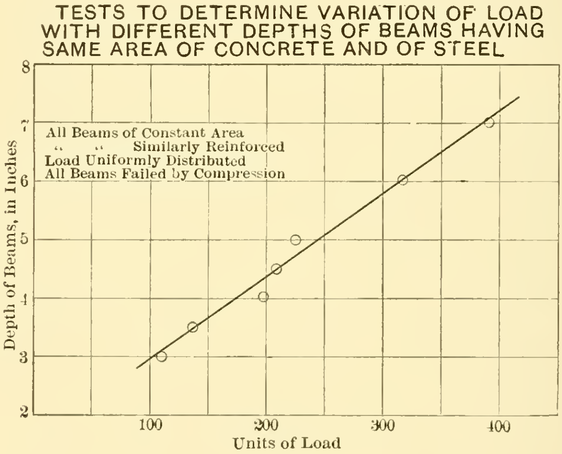

Fig. 21.

The analysis of the various test beams developed several different points, one of which was the fair constancy of a quantity corresponding with the h of the author's formula. Even the several hundred examples, however, were found to be altogether too few, upon which to base perfect reliance, and the speaker has thus been forced to resort largely to theory in order to develop working rules for design; but these rules are well tinctured with the deductions from the experiments analyzed.

The author's work, in analyzing the Watertown compression tests to arrive at a proper assumption as to the stress-strain curve, is in direct line with what he states must be done as a solid basis upon which to construct working formulas. It is an indirect method, but, nevertheless, of great value.

Mr. Goodrich.

His objection, that the assumption of a right line as the stress-strain curve for concrete gives beams which are unnecessarily heavy, may be true from the point of view of theory only, and perhaps may be proved so in practice when only the best of workmanship is exacted. However, when consideration is given to the fact that probably a large majority of the reinforced concrete work going on at the present time is being done by inexperienced contractors, employing relatively low-grade labor, and under little or no supervision, the speaker believes that the Building Department of the City of New York, the Prussian Government, and the French Commission have taken a wise course in specifying the right-line assumption. In the example given by the author, he shows an increase of only ½ in. for a beam which would have a depth of 8 in., according to his method of design. This is an addition of only 6¼%, which is very small to cover even ordinary variations in workmanship, and is only 3¾% more than enough to cover the variation from the nominal weight of steel bars allowed in the rolling mills, according to standard steel specifications.

On the basis of concrete at 20 cents per cu. ft., steel at 3 cents per lb., and 1% of steel used as reinforcement, a variation in cost of only 3.6% is found between the right line and the author's assumptions. While bars usually run over weight, still there is a real chance of a possible deficiency, in the carrying power of a beam, of as much as 2½%, against which safe practice recommends a constant increased cost of only 3.6 per cent.

What the author says as to the proper definition of the modulus of elasticity of concrete is only too true. The speaker has been unable to find any data as to the modulus for concrete 14 days old, which is the age at which he determines all working values, and has been forced to assume one until such time as accurate tests can be secured. He has lately come to the conclusion that the usual ratios assumed for the moduli are too small.

Fig. 22 shows the curves representing the theoretical percentage of depth from the top of a beam to the neutral axis, for different percentages of steel and ratios of moduli, based on a right line for the stress-strain diagram. On it are also plotted the actual locations of the neutral axis determined by measurements on the beams tested by A. N. Talbot, M. Am. Soc. C. E., and reported to the Western Society of Engineers. It is seen that the curve coming nearest to the observed results would be for approximately 18, as a ratio of moduli.

Another illustration, tending to prove this conclusion, was of a test beam designed with stirrups of the special type adopted by the speaker. When only 14 days old it failed by compression under a center load of 16 000 lb. The effective beam area was 7 in. wide by

Mr. Goodrich. 10 in. deep, and the clear span was 100 in. On the assumption of a right line for the stress-strain diagram, and a ratio of 12 for the ratio of moduli, the extreme fiber stress at failure was 3 800 lb. With a ratio of 20, the fiber stress was 2 750. This means one or both of two things: the ratio of moduli assumed in the first case was altogether too small, or the type of stirrup had much to do with raising the extreme fiber stress. Probably both are true. These points, with many others, make the speaker feel that values for r below 15 are really too small. This is most likely to be true with certain designs of web reinforcement which act to stiffen the beam.

The speaker agrees with the author when he says that the "most logical" method of design is to compute the load producing the

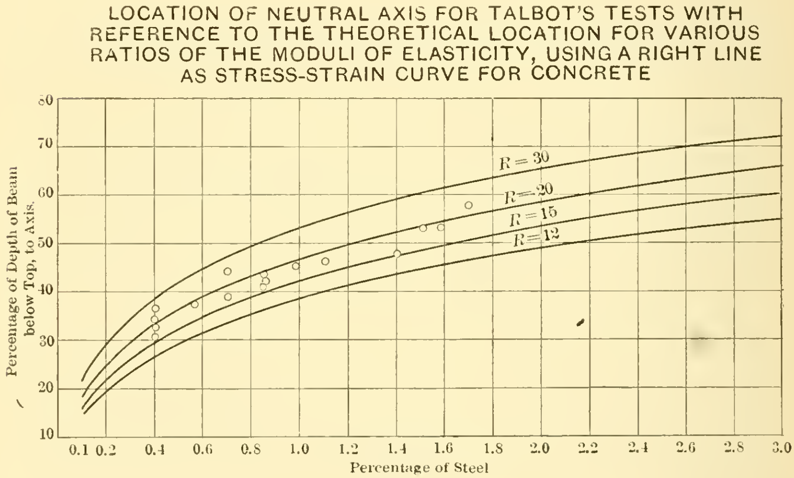

FIG. 22.

maximum stress "as the dead load, plus the product of the working live load by a factor of safety." That is the method adopted by the speaker for all his work.

It is interesting to note that, after all his work, the author finds that the average values of his quantity, h, vary only from 0.865 to 0.84, even with the widest range of theory as to the form of the curve in question. When he assumes a constant value of 0.85, he strikes almost exactly the one found for a steel stress of 40 000 lb. per sq. in. Both 40 000 and a value very close to 0.85 are the ones adopted by the speaker for all his ordinary work. In selecting 40 000, he agrees with the author as to adopting the elastic limit of the steel as one maximum stress.

The speaker is also in substantial agreement with him as to the propriety of adopting, as a working value for the maximum stress for concrete, an amount reduced appreciably below the ultimate stress usually assumed for that material. The author, like most engineers, adopts 2 500 as the ultimate stress. He, however, reduces this amount by 20% and uses 2 000 as the maximum allowable stress on concrete. The speaker uses this same figure, but arrives at it by a different course of reasoning. Mr. Goodrich.

It is believed that almost all structural work, both of steel shapes and of reinforced concrete, is likely to receive its most severe test during erection, and that (in the case of concrete at least) it is most apt to occur about 14 days after the concrete has been placed. The centers are then removed or are being removed, and the structure is in such a condition that building materials and all sorts of things are likely to be piled about in the promiscuous way so well known to all engineers. Little thought is ever given to the actual weight involved, and overloading is known to be only too common.

The diagram found on page 257 of Taylor and Thompson's book, "Concrete, Plain and Reinforced," shows that the average ultimate compression stress in concrete 14 days old is about 85% of its ultimate stress at the age of one month. This tends to show that the reduction of 20% advocated by the author is a wise one, especially when conditions are considered which are apt to occur at an early age of the concrete.

For these reasons the speaker has prepared all his working formulas on a basis of 2 000, which is lower than has generally been used by most engineers, except through the devices of a relatively low safe working stress for concrete, or of a larger factor of safety for concrete than for steel.

The author suggests that experiment is best to determine "the maximum allowable percentages of steel for each grade of concrete." In this connection, the deductions made by Mr. Condron are of interest. He says:

"For plain steel bars of approximately 33 000 lb. per sq. in. elastic limit..........*P* (is) not to exceed 1.5%," and for "corrugated or similar bars of approximately 55 000 lb. per sq. in. elastic limit......$1\frac{1}{4}$ per cent."

He does not give his reasons for the deductions, and the method of loading, and size and kind of reinforcement in the beam influence so largely the type of failure that some modification of such a general rule should be made, so as to meet varying conditions.

In this connection, the results of the tests made by H. A. Carson, M. Am. Soc. C. E., published in the report of the Boston Transit Commission for 1904, are of great interest. Fig. 23, at the left, shows the plotted results of tests of plain, square and other bars of

Mr. Goodrich. low elastic limit. In all cases the failures are described as being due to "tension," except two with small percentages, in which one bar slipped. Fig. 23, at the right, shows results obtained with twisted and corrugated bars. In the tests marked with double circles, the beam failed by what the report designates as "shear," but which the speaker believes to be due primarily to a splitting of the concrete along the line of reinforcement because of the shape of the bar and its action when it begins to draw through the concrete as tension comes upon the reinforcing steel.

Since the beams reinforced with plain rods did not fail by shear or by slipping of the rods, even when the percentage of reinforcement was high, there is nothing to lead one to presuppose failure by shear in the cases of the beams with twisted and corrugated bars.

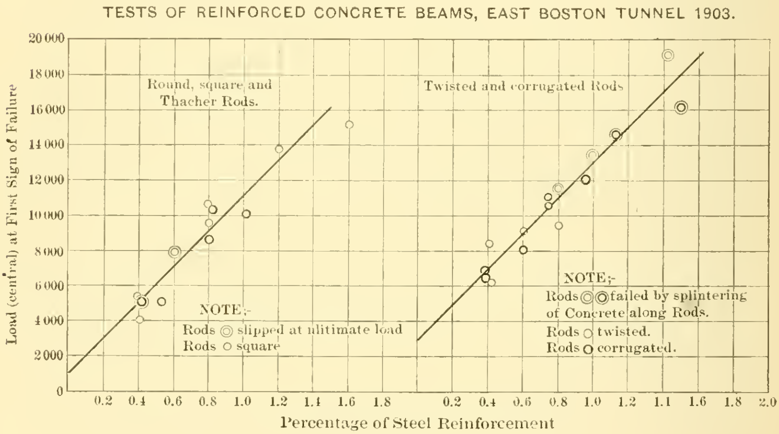

Fig. 23.

Fig. 1, Plate XXVII, is a reproduction of one of several photographs contained in the report, to which reference is specifically made as illustrating the type of failure in these cases. It may be contended that the bars were spaced too closely, so that not enough concrete surrounded the bars to bring them into proper action. This is doubtless true, but when sufficient concrete is available for this purpose, the design is a failure from the economical standpoint.

The tests made by Mr. Carson show that—for beams of the size involved, reinforced simply by tension bars, and broken by a center load—not more than 1% of any but plain steel bars should be used. It will be seen, later, that when stirrups are used, much larger percentages of steel can safely be utilized.

The analytical work carried out by the author is extremely interesting in itself, but it is even more so when it is examined as to its agreement or disagreement with actual tests. Mr. Goodrich.

The speaker has investigated a great many theoretical designs based on almost every combination of conditions, and one set of results is shown in Fig. 24. The depths of beams of constant width, designed to support a given total load on a given span, with varying percentages of reinforcement, were computed. One curve was plotted showing the varying depth when only the steel was considered. Another curve was then found when the concrete alone

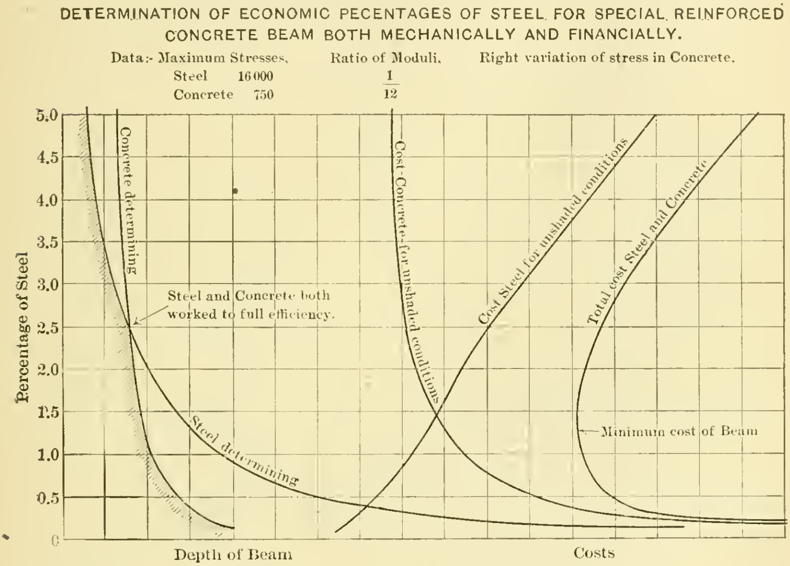

FIG. 24.

was kept in view. These two curves are at the left in Fig. 24, and where they cross is the only point at which both materials are worked to their full mechanical efficiency.

With a percentage of steel other than that found at the point of intersection of the curves, the depth is determined by one or other of the unshaded parts of the lines. The costs of both steel and concrete for beams of the depths determined by these unshaded lines were then computed, and are shown in the next two intersecting curves. The total cost is given by the line at the right, which shows a decided minimum. In this special case, however, the cost varies only slightly between 1 and 1½% of steel.

Mr. Goodrich. With other assumptions as to the ratio of moduli of the maximum allowable stresses, and of the unit costs of the materials, the minimum occurs with other percentages of steel.

Some actual plotted costs are shown in Fig. 25, which includes Professor Talbot's tests with corrugated bars, and the Boston tests with square and with twisted bars. The minimum occurs between 1 and 1½% of reinforcement for the special prices assumed, *viz.:*

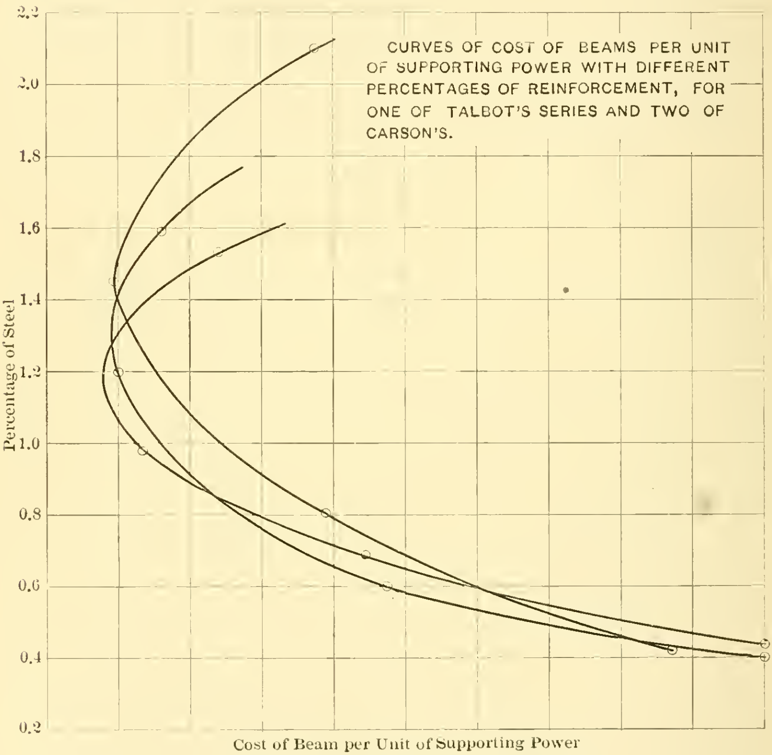

FIG. 25.

20 cents per cu. ft. for concrete and 3 cents per lb. for steel. These are the cost prices assumed by the author.

With respect to possible economies which might be effected by introducing steel in the compression edge of the beams without using it in the web also, only the tests for the Boston Tunnel shed any light, as far as the speaker is aware. However, they entirely bear out the author's deduction, that "single reinforcement" will be

cheaper. The left-hand portion of Fig. 26 shows the results obtained, and little increase in supporting power is observable, probably because all beams will fail, by so-called "shear" in most cases, long before the top reinforcement rods can be brought into action, unless special means are provided to make them do so. Mr. Goodrich.

From some experiments on the longitudinal reinforcement of columns which the speaker has seen, he feels certain that, in both beams and columns, steel designed to take compression, under such conditions, carries but a small percentage of what it is usually calculated to do. If it is necessary to increase the compression side of a girder with steel, it can be done much more economically in other ways, the beam described earlier, which broke with an extreme fiber stress of more than 2 750 lb. per sq. in. tends to prove this.

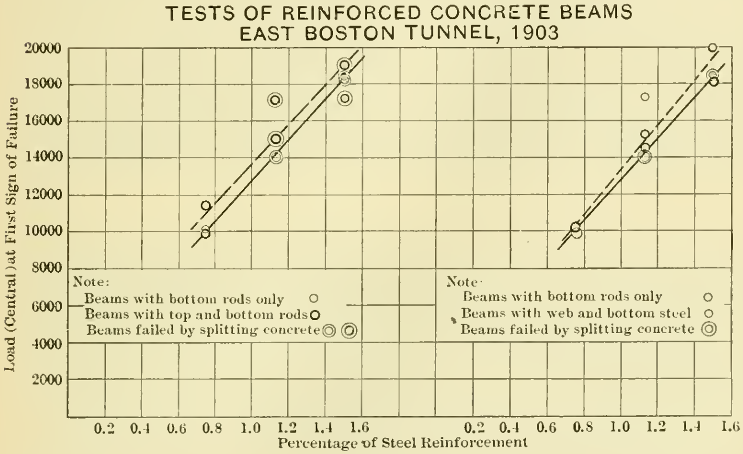

Fig. 26.

The author's paragraph with regard to the ideal web reinforcement should be fruitful of interesting discussion. Personally, the speaker does not agree with most of the clauses contained in it. However, he believes thoroughly in web reinforcement, and that it should consist of a multiplicity of small members which should extend to the top of the beam. The experiments made for the Chicago, Milwaukee and St. Paul Railway by Mr. J. J. Harding, and reported to the Western Society of Engineers, strongly tend to prove each of these items, but especially the first and last; while other experiments witnessed by the speaker some years ago led him to the conclusion contained in the second point, to which he called attention in his discussion of Captain Sewell's paper* before the International En-

*Transactions, Am. Soc. C. E., Vol. LIV, Part E, p. 459.

Mr. Goodrich. gineering Congress. In the other points, issue is taken with this paper; however, these matters are not necessary to the analytical treatment, the results of which are believed by the speaker to be essentially correct and to be borne out by experiment.

In exact accord with the author's ideas, the speaker designs beams primarily with regard to the maximum moments, without reference to anything except an approximate dead load. The latter is usually taken as 1 lb. per sq. in. of "guessed-at" cross-section per foot of length. Afterward, web reinforcement is added, and all the parts thus determined are investigated as to other possibilities of failure.

The best comparative experiments known to the speaker, illustrating the value of web reinforcement, are those made by Mr. Harding, to which reference has been made. He found that his web reinforcement increased the carrying power of the beams which contained it about 50%, and that the variation in strength was much less in beams with web reinforcement than in those without it. Mr. Harding experimented with only a single percentage of steel. In some tests for the Boston Tunnel, a wide variation in the percentage of tension reinforcement was made. The same relative effect of web reinforcement was observed, but no results as striking as those of Mr. Harding were obtained. In the curves shown at the right of Fig. 26, beams with bottom rods without web steel are compared with those with bottom rods and with vertical web reinforcement. The value of the latter will never be apparent until the load has reached a point beyond which the concrete cannot withstand the developed stresses. In Professor Talbot's discussion of Mr. Harding's paper, this value is given as:

Vertical shear $\div$ (breadth $\times$ effective depth) $=$ from 125 to 150 lb. per sq. in.

It is well known that when conditions are right, concrete can develop a very large shearing resistance, and the speaker believes that properly designed beams should take account of this capacity. In this point he differs from the author, who states that "ideal web reinforcement" should be of such size that "the sum of the horizontal components of the stresses in all the web members in each half of the beam should be at least equal to the maximum stress in the flange reinforcement." The speaker believes that this would give an excessive amount of web steel.

Turning to the Boston experiments, it is seen that the beams provided with web steel show little improvement over beams without it, until a point is reached above which the latter beams fail by "shear"; but, even then, only small improvement is found, due probably both to the design of the web reinforcement and the relative depths of the beams.

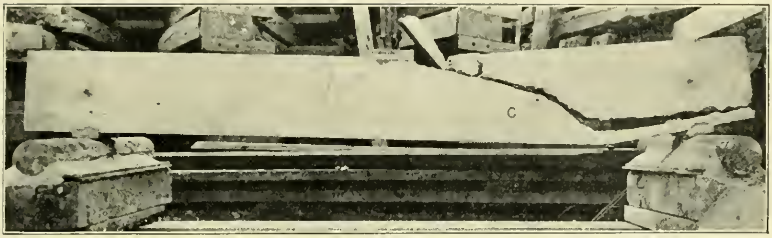

Fig. 1.—Beam tested for Boston Tunnel, without Stirrups, showing Type of Failure.

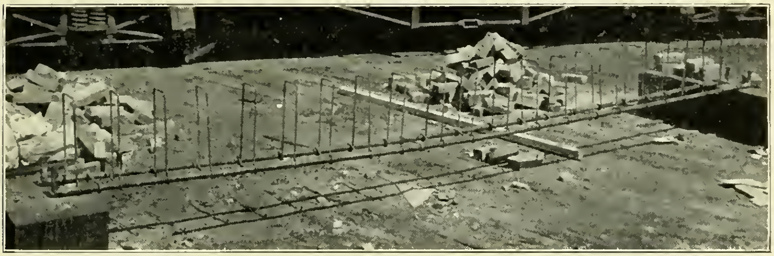

Fig. 2.—Reinforcement used in Brick Beam Test.

Fig. 3.—Brick Beam, not loaded.

Fig. 4.—Brick Beam supporting Load.

When comparison is made between beams which have both top and bottom rods, with and without web reinforcement, or beams which have web reinforcement with and without top rods, a decided improvement is apparent. The results of such comparisons are shown in Fig. 27, in which, except where stated to the contrary, loadings are given at the first sign of failure. When ultimate loads are considered (at which the speaker usually looks askance), a still more striking improvement is apparent. It should also be noted Mr. Goodrich.

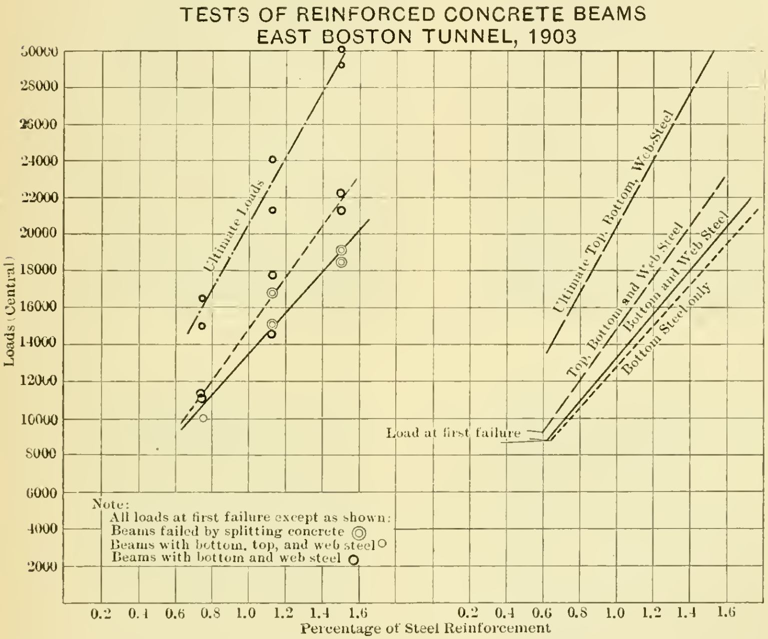

Fig. 27.

that the results given for Mr. Harding's experiments refer to loads at ultimate failure.

Unless a large percentage of tension steel is used, and the beams are comparatively deep, it will not usually be found financially profitable, purely from the point of view of increased safe load capacity, to use web reinforcement; and, even with large percentages of tension steel, it is rarely profitable unless the increase in the ultimate loads is noted, and advantage is taken to reduce the required factor of safety accordingly.

But, laying aside all ideas of economy, the speaker believes emphatically in the use of ample web reinforcement, because of the

Mr. Goodrich. marked difference in the type of failure which will take place. Fig. 1, Plate XXVIII, shows a beam from the Boston series which was reinforced with bottom and web steel and carried more than 19 000 lb. at the first sign of failure. Even when ample factors of safety are used, it is well worth the cost of web reinforcement to be sure that when failure does occur it will be like Fig. 1, Plate XXVIII, rather than like Fig. 1, Plate XXVII.

It was from a study of such conditions as are shown in the case of top rods with web steel that the speaker was led to adopt, for his work, a type of web reinforcement which did not depend for its action on its adhesion to the concrete. That is the only means that most systems have of transmitting stresses from the web steel into the concrete, whether the web steel takes the form of straight spines, or consists of vertical or other shaped members primarily designed as aids to easy construction. A much better form, in the speaker's opinion, is one in which the web steel is actually bound to the floor slab reinforcement. In this case, however, the latter should run across the top of the beam. But still better for construction purposes is a design in which the tension rods are placed in pairs, and the web steel is shaped like an inverted U with the free ends wrapped around the tension rods. The speaker claims no originality for this design, although he believes he was among the first to see clearly its mechanical and constructional advantages, and first worked out its proper design. By this system, the web steel may or may not be rigidly connected to the tension rods. In some beams, rigid connection is well, especially where the reinforcement is fabricated at a point distant from the point of installation, and the amount of handling during shipment is large.

The speaker must include himself among those to whom the author refers, and who cannot agree with him that "attached web members are necessary." The speaker believes that the two following tests, devised and carried out under his direction, go far toward disproving the need of the rigid attachment of web members.

A "truss" was built, consisting of tension bars hooked at the ends and of inverted U-shaped stirrups with their ends simply wrapped around the tension rods. The stirrups were spaced so that ordinary hard building brick could be placed between them. Flat plates were set in front of the bent ends of the tension rods to prevent the end bricks from being cut by the rods, and oak spacing pieces, of just the thickness of the stirrups, were used to separate the bricks which would otherwise rest against the stirrups and be cut by them. Figs. 2, 3 and 4, Plate XXVII, show the plain reinforcement, the brick beam, and the load carried when the top bricks began to crush, respectively.

A more interesting experiment lately carried out by the speaker was the following:

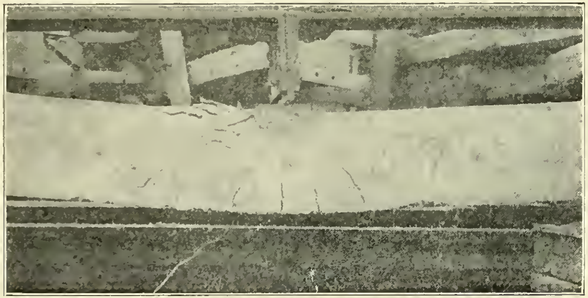

FIG. 1.—BEAM TESTED FOR BOSTON TUNNEL, WITH STIRRUPS AND BOTTOM RODS, SHOWING TYPE OF FAILURE.

FIG. 2.—BEAM, 14 DAYS OLD, TESTED BY WRITER. FAILED BY PULLING OUT OF HOOK. NO STIRRUPS USED.

FIG. 3.—BEAM, 14 DAYS OLD, TESTED BY WRITER. FAILED BY COMPRESSION. NO CONCRETE AROUND TENSION RODS. STIRRUPS USED.

FIG. 4.—SAME BEAM AS FIG. 2. TIPPED UPSIDE DOWN TO SHOW ATTACHMENT OF STIRRUPS, AND ABSENCE OF ALL ADHESION.

A concrete beam was moulded which was reinforced with a truss similar to that used in the brick beam. However, the concrete was not allowed to get below the bottom rods except at the points of support. In this way no possibility of any adhesion between the tension rods and the concrete existed. Furthermore, there was no rigid connection between the bottom rods and the stirrups, as the latter were simply wrapped around the former. At an age of 10 days the beam failed by crushing at the center, with a load of 9 326 lb., while a similar beam, built without stirrups, failed under a center load of 9 000 lb. by rods pulling out of the concrete. Fig. 2, Plate XXVIII, shows the latter beam, and Fig. 3, Plate XXVIII, shows the failure of the beam which had stirrups, but had no concrete under the tension rods. Fig. 4, Plate XXVIII, shows the beam tipped upside down after it had been broken, so as to show the method by which the stirrups were attached to the tension rods. Mr. Goodrich.

The design of web reinforcement adopted by the speaker also materially increases the resistance which the beam will develop to compression, as it acts in a manner similar to that of the hooping or spiral steel used to reinforce columns. Moreover, the increased crushing strength is developed in both beams and columns, with only a fraction of what is required where longitudinal rods are used, as the tests of Considère tend to prove.

The test described earlier in this discussion, in which was developed an extreme fiber stress of from 2 750 to 3 700 lb. per sq. in. for concrete 14 days old, is a strong proof of the advantage claimed.

Whenever possible, of course, recourse is had to the device of using the floor structure adjoining the compressed edge of beams to increase the compression area of the beam, but the speaker has never been satisfied as to the accuracy of the methods of computation thus far published for such beams. It is something of a disappointment that the author did not give some analysis of the relations which should exist between his quantities, *s*, *b* and *c*, in **T**-beams. He takes for granted a predetermined relation which is not involved in the determinations of economic dimensions, and perhaps he is right. In this point, it is concluded that he uses the same methods of determining the quantities in question as he gave in his paper before the International Engineering Congress, in which he followed A. L. Johnson, M. Am. Soc. C. E. It is a great pity that so few experimenters have tested beams of **T**-section, because they are the ones oftenest encountered, and every designer makes large use of the flanges of the **T** in his designs.

The analysis of the several hundred beams examined by the speaker has convinced him that reinforced concrete beams designed with vertical stirrups like those shown in Fig. 2, Plate XXVII, may

Mr. Goodrich. be analyzed like a multiple-system Howe truss, as far as most points in the design go. He is borne out in this idea by three tests of full-sized members which were executed during the past year in connection with the construction work going on under his direction.

The floors for one building were designed to carry a load of 800 lb. per sq. ft. at the first crack, when 30 days old. Two full-sized modified T-beams were broken at an age of 14 days. They were designed for the building work as partially anchored at the ends, so that the denominator in the moment formula was 10 instead of 8, as for beams simply supported at the ends, and as the tests necessarily must be. Rails were loaded upon the test beams, and their weight was sufficient to cause failure without the possibility of arching or of other troublesome effects. The actual load safely carried by the tests when the first cracks appeared, when increased in the ratio of 10 to 8, was equivalent to a load of 815 lb. per sq. ft.

Four months after one floor of the building had been completed, four full bays, aggregating a total area of about 1 000 sq. ft., were loaded with brick to aggregate 875 lb. per sq. ft. Under this load, a fine crack appeared at the center of the bottom of the two fully-loaded girders, with no signs of strain visible at any other point except small deflections. Great care was taken to pile the brick so as to prevent any arching effect, and, as the greatest deflections were slightly more than $\frac{1}{4}$ in. for 17-ft. spans, little reduction of load from arching could have occurred in any case.

The third test was of a beam designed to carry a central load, and illustrates two important points, in spite of the fact that the concrete was of very inferior quality, for some reason as yet unfathomed by those in charge. Figs. 1 and 2, Plate XXIX, show the beam as a whole, and a near view of the principal failure. Just as the last rail was applied to the test, tension cracks appeared near the center of the beam. At almost the same time, spalling of the concrete took place at the top, at the center. This might have been hastened by the action of the load, but only slightly so, as good supports were provided to distribute the load properly. Before another rail could be added, the failure shown in the photograph took place. It is one in which a whole panel between two of the vertical stirrups seemed to shear out (using the word in its proper sense). There was no indication of failure by diagonal tension, according to the usually accepted idea of shear; and the failure cannot be attributed to horizontal shear, as the structure of the exposed concrete shows that the stresses were solely vertical ones. In this connection, it should be noted that most designers give the horizontal steel its full value in resisting vertical shear. It is believed that this method of design is proved to be of doubtful value by this experiment.

One point, clearly brought out, is that a type of failure, which

FIG. 1.—BEAM WHICH FAILED BY TENSION, COMPRESSION AND END SHEAR, SIMULTANEOUSLY.

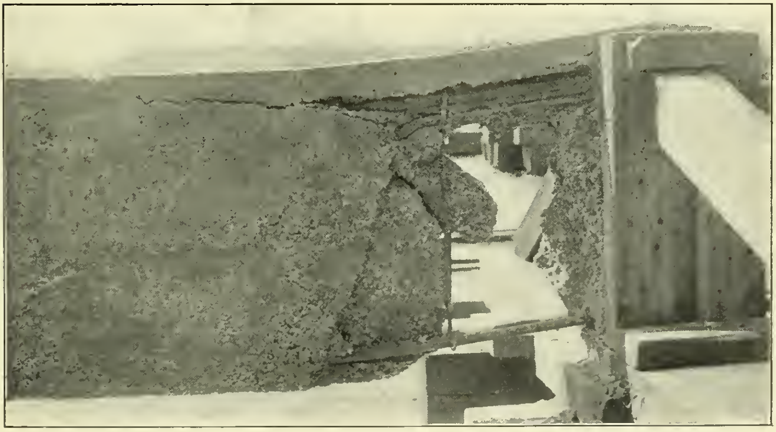

FIG. 2.—VIEW SHOWING CHARACTER OF FAILURE DUE TO END SHEAR.

has not generally been considered, is possible when numerous stirrups are used. There was no pulling out of the bottom rod from the concrete at the point of support, and there was perfect action between the concrete and the unattached stirrups in this case. The failure occurred in the first panel away from the abutment, and at a load far less than the longitudinal steel would carry theoretically in shear. Mr. Goodrich.

One special point, from which the speaker draws some consolation with regard to this test, is that the quantities assumed in making up the design were so well balanced that the three principal types of failure showed themselves almost simultaneously. This shows that all parts of the structure were consistently proportioned and that, with safe loads, all parts would have equal factors of safety. Of course, with better concrete, this relation would not be quite so close, but then failures would have taken place through tension, and the other parts would have been very near failure as well. Of all types, a failure by tension is least to be feared, as the author intimates. However, only in beams with very low percentages of reinforcement will failure occur without a considerable crushing occurring after the steel has passed its elastic limit.

The speaker believes that the type of reinforcement he has adopted is far cheaper than that advocated by the author, and will be even more efficient in some cases. The experience of the past year, during which several thousand tons of such reinforcement were fabricated and worked into place under the speaker's direction, has shown that steel fabricated and ready for placing costs only about $45 per ton, and that the cost of placing in the forms is less than ¼ cent per lb.

When the author discusses the advantages of rigid attachment of web members, in regard to fire tests, he makes a statement about "web members wrapped around the tension bars" being inadequate. To this the speaker takes exception, and he feels that the test of the concrete beam illustrated in Figs. 3 and 4, Plate XXVIII, tends to prove the fallacy of the author's statement.

The speaker thoroughly agrees with the author in all that he says in his conclusion about the advantages of having web reinforcement run quite to the tops of beams, but he would substitute the word "vertical" for "diagonal" in each case.

The speaker has been intensely interested in this subject for a long time, but feels that there is more experimental work available for analysis than the author admits in his last paragraph.

This opportunity is taken to give expression to a gradually growing feeling, on the speaker's part, that much of the analytical work now in vogue is based on wrong primary assumptions. At least one writer of a book on the subject of reinforced concrete has hinted at

Mr. Goodrich. the same thing, and an engineer of some eminence has lately broached the same idea in a paragraph of a public address which has been printed in an engineering periodical.

In any case, the thanks of the profession are certainly due to Captain Sewell for his timely and scholarly paper.

Mr. Thacher. EDWIN THACHER, M. AM. SOC. C. E. (by letter).—The writer has no doubt that the author's Equation 0 will give as reliable results as any other formula, however complicated, provided the values, h and A, are well established.

As noted by the author: "A must be expressed as some fraction of the area of concrete." A should bear such relation to d that the strength of the concrete in compression is equal to the strength of the steel in tension. If, in Equation 0, the values of A and T be substituted, it becomes $M = h\ d^2$, which is simpler still, and this is the formula giving the moment of resistance of a rectangular concrete-steel beam which has been used quite extensively by the writer and others for the past six years. It was first published in the *Transactions* of the Association of Civil Engineers of Cornell University,* and, since then, has been published in *Cement*,† in *Engineering News*,‡ and in a pamphlet issued by the Concrete-Steel Engineering Company, and is quite well known. In the same publications, formulas are given for the ultimate strength of beams, supported at the ends and loaded at the center, in terms of $\frac{d^2}{l}$; for beams uniformly loaded, in terms of $\frac{d^2}{l^2}$; and for finding the depth of beam to sustain any required load, l, being known. Nothing simpler is possible or could be desired. They are in the same form, and require no more labor in calculation than formulas for wooden beams.

In the writer's formulas, the stress-strain line is considered as straight, but, if considered as a parabola or any other curve, the only change in the formulas will be in the value of the constant coefficient, and if this is established by experiment it does not matter what shape of stress-strain curve is usual in calculation.

In the formulas above noted the constant coefficients were determined theoretically. The ultimate strength of the concrete and steel were used; also such values of E_c as resulted from the highest pressures recorded. From the results of such tests as the writer has been able to work up, he has had no occasion to change the values of the coefficients. Five sets of tests, made in different localities, and by different men, cover a variation in composition of concrete from 1:2:4 to 1:3:6, a variation in age of specimens from 23 to 90 days, a variation in ratio of length to depth of 6.0 to 22.2, a variation in

* Vol. X, 1902.

† July, 1902.

‡ February 12th, 1903

strength of metal from 50 000 to 100 000 lb. per sq. in., and a variation in percentage of metal from 0.31 to 3.9 per cent. The maximum variation between actual and estimated strength in no case exceeded 16 per cent. The mean variation for any one set of tests in no case exceeded 2.3%, and the mean variation for all tests, 30 in number, was 0.013%, or practically zero. Mr. Thacher.

In 9 of the 30 tests the reinforcing bars broke. The writer does not see how any formula for ultimate strength of concrete-steel beams can be even approximately true when the elastic limit of the steel is used in the formula, unless the constant coefficient is modified to compensate therefor.

The elastic limit of steel is about six-tenths of its ultimate strength. Considère and Professor Bach state that the elastic limit of concrete, as far as concrete can have an elastic limit, is also about six-tenths of its ultimate strength in compression, so that it appears to the writer that if the author uses the elastic limit of steel in his formulas he should use 60% instead of 80% of the crushing strength of the concrete. The writer agrees with the author that it would be very desirable to make numerous tests of beams using, say, 1:2:4 and 1:3:6 concrete, and containing various percentages of steel reinforcement, for the special purpose of establishing the values of h in his Equation 0, or the coefficient of d^2 in the writer's formula. From the result of Professor Hatt's experiments, the writer drew the conclusion that if the amount of reinforcement is double what it should be to equal the strength of the concrete the gain in strength of beam due to lowering the position of the neutral axis will be about 20%, and if the value of E_c is doubled there will be a loss of strength of about 20 per cent. The writer agrees with the author that the safe loads should be determined by a factor of safety, for the breaking loads can be found by tests, and, for any intermediate loads, the value of E_c is constantly changing. He does not agree with him, however, regarding the factor of safety recommended, that is to say, nothing for dead load and 4 for live load. If there is anything that will ultimately bring concrete-steel construction into disrepute the writer believes it will be due to the reckless method of proportioning followed by some constructors. The writer never heard of a bridge or an arch being designed with a factor of safety of 1 for dead load, and the engineer who would undertake it would probably not repeat the experiment. The writer believes that if cost prohibits a factor of safety of at least 3 for both dead and live loads it will be better to use wood or some other cheap but safe construction. The investigations and formulas of the author regarding minimum cost are interesting and instructive, but the writer doubts whether they will find much practical application, as the depth of beams is frequently governed by practical considera-

Mr. Thacher. ations, and the minimum percentage of steel is fixed by calculation, and any greater amount than that is partially wasted. The writer does not agree with the author that the attachment of the web reinforcement to the horizontal reinforcement should necessarily be independent of the concrete. As long as the web reinforcement is attached to the flange reinforcement, and is securely clamped thereto by the concrete, no movement can take place, and all conditions are satisfied.

If, as the author considers, it is necessary to provide steel for all shearing stresses, it appears to the writer that all superfluous parts of the concrete should be removed, resulting in what is known as the Visintini system, which the writer considers a very excellent system, it being the application of the truss principle to concrete-steel construction.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

ANTHONY HOUGHTALING BLAISDELL, M. Am. Soc. C. E.*

DIED SEPTEMBER 9TH, 1905.

Anthony Houghtaling Blaisdell was born at Coeymans, Albany County, New York, on December 23d, 1848. He had some of the best Dutch blood in the country in his veins. On his mother's side he was descended from Colonel Anthony Van Bergen (an officer in the Revolutionary War) and on his father's side from Levi Blaisdell, of Amesbury, Mass., who entered the Revolutionary Army at sixteen years of age as Ensign, and who, at the close of the war, settled in Coeymans, where his descendants still possess a part of the old land grant from Charles I.

Mr. Blaisdell entered Rensselaer Polytechnic Institute, at Troy, New York, at the age of eighteen, and was graduated therefrom with high standing in the class of 1870. In September of that year he entered the service of the United States in the Engineer Department, with which, to a greater or less extent, he was identified during all his subsequent career. His work was largely upon the Mississippi and its tributaries, and, during the greater portion of the time, his headquarters were in St. Louis, Missouri. He rendered important service upon the construction of the Des Moines Rapids Canal, and surveyed and helped carry on improvements on several inland rivers west of the Mississippi. In particular, he was connected for many years with the improvement of the Missouri River and its important tributary, the Osage.

Mr. Blaisdell was an expert boat builder, especially skilled in the ironwork of snagboats and other craft connected with river improvement work, and several of these important boats were built from his designs.

During the greater part of his service with the Government he was the Principal Assistant of Colonel Charles R. Suter, Corps of Engineers, U. S. A. He also served with Colonel Amos Stickney, and to a less extent with other officers of the Corps of Engineers.

In 1879 Mr. Blaisdell went into the private business of boat building in St. Louis, under the firm name of Allen and Blaisdell, and continued in this business until 1885. Owing to various causes, this undertaking did not prove successful, and Mr. Blaisdell re-

*Memoir prepared by H. M. Chittenden, M. Am. Soc. C. E.

turned to the Government service in the latter year. While engaged in private business Mr. Blaisdell was a most active and useful citizen in both educational and philanthropic lines, serving for three years on the St. Louis Board of Education and leaving everywhere marks of his accurate and organizing mind, no detail being too small to be carefully weighed and its value determined.

In his professional career in the service of the United States Mr. Blaisdell was one of its most trusted employees. He was a man of absolute truthfulness and integrity of character, an earnest and industrious worker, loyal to his superiors, and, all in all, one of those men who make it possible for their employers to accomplish important work. Sincerity marked every act of his life. He was not an ambitious man, in the sense of endeavoring to reach beyond the positions which seemed naturally to fall to his lot, and, whether or not his situation promised him advancement, it made no difference in his fidelity to duty. He was thoroughly beloved by all his employers, and was never lacking a position in the Government service as long as his health permitted him to retain one.

In 1878 Mr. Blaisdell married Miss Mary McConnell, of Chicago, who still survives him. This union was blest with two children, one of whom, Mr. Robert Van Bergen Blaisdell, survives him.

Owing to declining health Mr. Blaisdell left the Government service in 1903 and returned to the parental estate at Coeymans, where he died in 1905, in the house in which he was born.

Mr. Blaisdell was elected a member of the American Society of Civil Engineers on March 3d, 1880.

GEORGE DRAPER STRATTON, Assoc. M. Am. Soc. C. E.*

Died November 21st, 1905.

George Draper Stratton was born in Orange, New Jersey, on June 5th, 1870. When he was one year old his father moved with his family to Newburgh, New York, where he was interested in the Washington Iron Foundry. On the death of his father, in 1876, his mother moved to Riverside, California, with her children.

At the age of 15 he was forced to leave school in order to help support his mother and sisters. This he continued to do until his mother's death, in 1887.

Mr. Stratton used the small share which he received from his mother's estate in obtaining a college education. He entered Stan-

* Memoir prepared by Charles D. Marx, E. M. Boggs, Members, Am. Soc. C. E., and R. M. Drake. Assoc. M. Am. Soc. C. E., from information furnished by Mrs. Stratton.

ford University in the fall of 1891, with the pioneer class, as a special student. By hard work he made up his entrance deficiencies, and was graduated with honor, in the course in Civil Engineering, in May, 1895.

He at once commenced the practice of his profession, and, after several minor engagements, entered the service of the Southern Pacific Railroad Company. He was Assistant Engineer on the Sacramento Division for one year, and was then made Roadmaster at Marysville. Here his health became undermined by malaria, and at the end of the year he was transferred to the Western Division of the Southern Pacific Railroad, with headquarters at Oakland, California. He remained there for six years, or until death called him, on November 21st, 1905. His record as Assistant Engineer and as Assistant Resident Engineer was excellent.

Mr. Stratton was married, on January 17th, 1899, to Miss Jeannie Gift. His wife and a daughter, four years of age, survive him.

Mr. Stratton was a devoted churchman. Two years before his death he became a vestryman of St. Andrew's Church, Oakland. By his beautiful example while occupying that office, he was a source of great help to many. Mr. Stratton was a Mason and a Knight Templar. Among his associates, both business and social, he was much beloved for his fine character and sweet temper. He was ever and always the same quiet, even-tempered man, whose sincerity and loyalty were unfailing. The profession has lost in him a good engineer; the world, a good man.

Mr. Stratton was elected an Associate Member of the American Society of Civil Engineers on October 4th, 1899.

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

April, 1906.

PROCEEDINGS = VOL. XXXII—No. 4

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CIVIL
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FOUNDED
1852

HERMAN W. SPOONER

Published at the House of the Society, 220 West Fifty-seventh Street, New York, the Fourth Wednesday of each Month, except June and July.

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OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL. XXXII. No. 4.

APRIL, 1906.

Edited by the Secretary, under the direction of the Committee on Publications.

CONTENTS.

NEW YORK 1906.

American Society of Civil Engineers.

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The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays, ourth of July, Thanksgiving Day and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER: - - - - 533 Columbus.
CABLE ADDRESS: - - - - "Ceas, New York."

Vol. XXXII. APRIL, 1906. No. 4.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

SOCIETY AFFAIRS.

CONTENTS:

MINUTES OF MEETINGS.

OF THE SOCIETY.

April 4th, 1906.—The meeting was called to order at 8.40 P. M.; Vice-President Emil Kuichling in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 152 members and 38 guests.

The minutes of the meetings of March 7th and 21st, 1906, were approved as printed in *Proceedings* for March, 1906.

A paper, entitled "The Panama Canal," by A. G. Menocal, M. Am. Soc. C. E., was read by title, and the Assistant Secretary presented some additional written remarks by the author, and also written communications on the subject by Messrs. Clemens Herschel and George B. Francis. The paper was discussed further by Theodore Paschke, M. Am. Soc. C. E., and the author.

Ballots for membership were canvassed, and the following candidates elected:

As Members.

Carl Gustaf Frosell, Pencoyd, Pa.
Alexander Haring, New York City.
Cicero Demerit Hill, Chicago, Ill.
William Stephen Menden, Brooklyn, N. Y.
John Franklin Skinner, Rochester, N. Y.
William Fish Williams, New Bedford, Mass.

As Associate Members.

Augustus Rowley Archer, Jersey City, N. J.
Calvin Lewis Barton, New York City.
Horace Corey Booz, Philadelphia, Pa.
Charles Carroll Boyer, Kirbyville, Tex.
Wilfred Atherton Clapp, Portland, Me.
Charles Frank Class, Camden, N. J.
John Edward Conzelman, St. Louis, Mo.
Farley Gannett, Harrisburg, Pa.
Adelbert Andrew Henderson, Pittsburg, Pa.
Frank Scott Howell, Ellis Island, N. Y.
John Imboden Hudson, Portsmouth, Ohio.
Howard Elmer Hyde, Manila, Philippine Islands.
Lindsey Louin Jewel, Norfolk, Va.
Adolph Judell, Toana, Nev.
Henry Jacob Kolb, Brooklyn, N. Y.
Earl Brink Lovell, New York City.
George Franklin Pawling, Philadelphia, Pa.
William Hendry Prentice, Jr., Muskogee, Ind. T.
Benson Bulkeley Priest, New York City.
Arthur Edwin Roberts, New York City.
Henry Chandlee Turner, New York City.
Fred J Wagner, Sylvan Beach, N. Y.
George Roy Wood, Pittsburg, Pa.

As Associate.

Moise De Leon, Atlanta, Ga.

The Assistant Secretary announced:

The transfer of the following candidates, by the Board of Direction, on April 3d, 1906:

From Associate Member to Member.

Edward Fulbister Kenney, Philadelphia, Pa.
Elmer Wayland Ross, Providence, R. I.
Harry Kent Seltzer, Austin, Tex.
Walter Mickle Smith, Dover, N. J.
James Knapp Wilkes, New Rochelle, N. Y.

The election of the following candidates, by the Board of Direction:

As Juniors.

On March 6th, 1906:

Harry Clifford Ford, Portsmouth, Va.
Pusey Jones, New York City.
Clifford Marshall King, Rupert, Idaho.
Ralph Ashur Pike, Mt. Vernon, N. Y.
Charles Albert Augustine Steegmuller, Long Island City, N. Y.
George Linton Watson, Philadelphia, Pa.

On April 3d, 1906:

Fred Edward Caldwell, Newton, N. J.
Charles Lee Doe, Elizabeth, N. J.
William Sheppard FitzRandolph, New York City.
Frederick Hall Fowler, Slippery Ford, Cal.
Henry Le Roy Fryer, Trenton, N. J.
Walter Leroy Huber, San Francisco, Cal.
John Aspin Kienle, Wilmington, Del.
Daniel Bernard O'Brien, New York City.
Charley Evans Shipman, Billings, Mont.
Ford Cushing Smith, New York City.
Edward Lee Soulé, San Francisco, Cal.
Samuel Wilson, Olean, N. Y.
George Creighton Wright, New York City.

The Assistant Secretary announced the following death:

Dunkin Wirgman Hemming, elected Associate Member, September 7th, 1892; Member, May 31st, 1904; died March 22d, 1906.

Adjourned.

April 18th, 1906.—The meeting was called to order at 8:40 P. M.; President Frederic P. Stearns in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, 97 members and 20 guests.

A paper by L. J. Johnson, M. Am. Soc. C. E., entitled "A Complete Analysis of General Flexure in a Straight Bar of Uniform Cross-Section" was presented by the author.

The Assistant Secretary announced the following deaths:

HENRY WILLIAMS PARKHURST, elected Member September 5th, 1877; died April 7th, 1906.

FREDERICK APPEL HAUSMAN, elected Junior February 3d, 1903; died March 6th, 1906.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

April 3d, 1906.—President Stearns in the chair; T. J. McMinn, Assistant Secretary, acting as Secretary; and present, also, Messrs. Bissell, Bowman, Gibbs, Gowen, Kuichling, Noble, Schneider, and Swensson.

The territory occupied by the membership of the Society was divided into seven geographical districts, as required by Art. VII, Sec. 1, of the Constitution.*

Applications were considered, and other routine business transacted.

Five Associate Members were transferred to the grade of Member, and thirteen candidates for Junior were elected.†

Adjourned.

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* See page 142. † See page 139.

ANNOUNCEMENTS.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

MEETINGS.

Wednesday, May 2d, 1906.—8.30 P. M.—A regular business meeting will be held. Ballots for membership will be canvassed, and a paper, entitled "The Control of Hydraulic Mining in California by the Federal Government," by William W. Harts, M. Am. Soc. C. E., will be presented for discussion.

This paper was printed in *Proceedings* for February, 1906.

Wednesday, May 16th, 1906.—8.30 P. M.—At this meeting a paper, entitled "The Scranton Tunnel of the Lackawanna and Wyoming Valley Railroad," by George B. Francis and W. F. Dennis, Members, Am. Soc. C. E., will be presented for discussion.

This paper was printed in *Proceedings* for March, 1906.

Wednesday, June 6th, 1906.—8.30 P. M.—A regular business meeting will be held. Ballots for membership will be canvassed, and a paper, entitled "Disposal of Municipal Refuse, and Rubbish Incineration," by H. de B. Parsons, M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings*.

Wednesday, September 5th, 1906.—8.30 P. M.—A regular business meeting will be held. Ballots for membership will be canvassed, and a paper, entitled "Concerning the Investigation of Overloaded Bridges," by Wilbur J. Watson, M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings*.

ANNUAL CONVENTION.

The Thirty-eighth Annual Convention of the Society will be held at The Hotel Frontenac, Thousand Islands, New York, on June 26th to 29th, 1906.

The general arrangements for the Convention are in the hands of the following Committee:

CHARLES S. GOWEN,
JOHN W. ELLIS, MORRIS R. SHERRERD,
J. WALDO SMITH, CHAS. WARREN HUNT.

NOMINATING COMMITTEE.

Under Article VII, Section 1, of the Constitution, the Board of Direction has divided the territory occupied by the membership into seven geographical districts for the purposes of the Nominating Committee, and now announces this division to the membership.

A map showing the new subdivision of the territory is printed on the opposite page.

District No. 1.—The territory within fifty miles of the Post Office in the City of New York.

District No. 2.—The States of Maine, New Hampshire, Vermont, Massachusetts, Rhode Island, and Connecticut (except as included in District No. 1), and all countries in Europe and Africa.

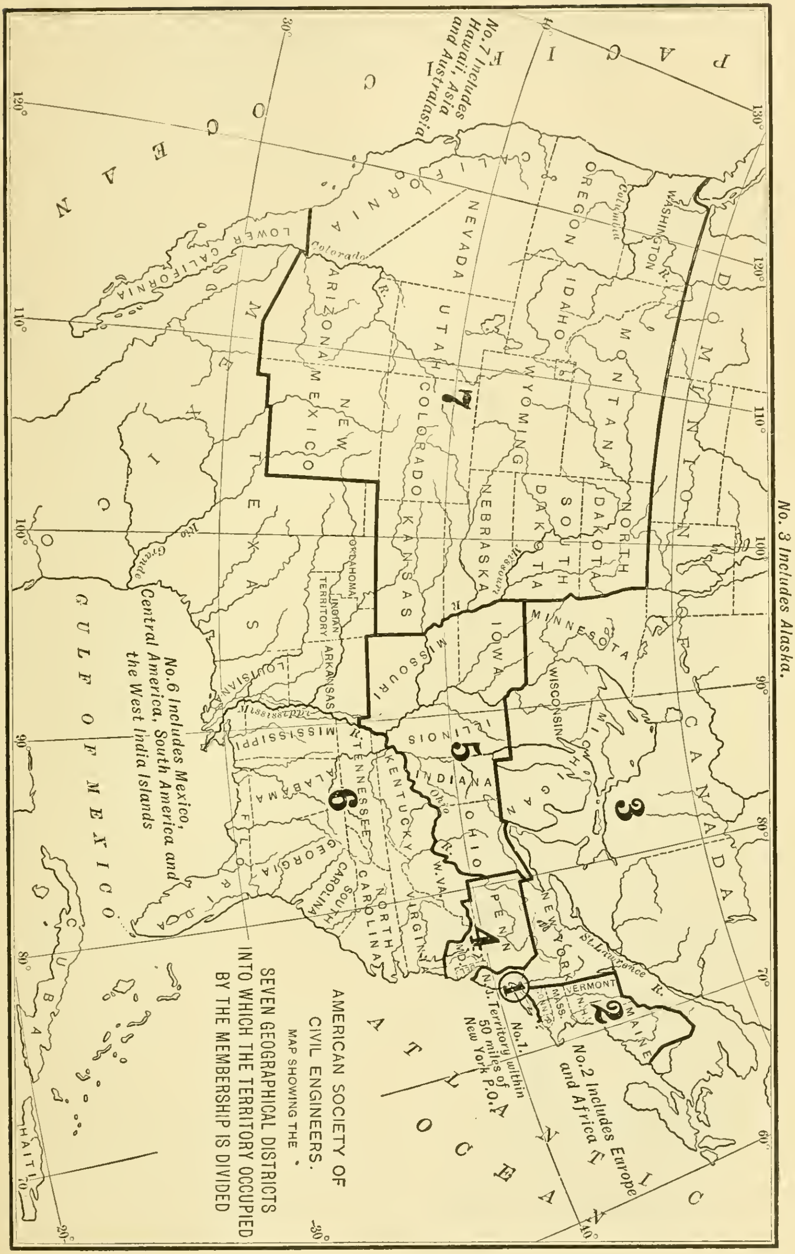
District No. 3.—The States of New York and New Jersey (except as included in District No. 1), the States of Michigan, Wisconsin, and Minnesota, the Territory of Alaska, and the Dominion of Canada.

District No. 4.—The States of Pennsylvania, Delaware, and Maryland, and the District of Columbia.

District No. 5.—The States of Ohio, Indiana, Illinois, Iowa, and Missouri.

District No. 6.—The States of Virginia, West Virginia, North Carolina, South Carolina, Georgia, Florida, Kentucky, Tennessee, Alabama, Mississippi, Arkansas, Louisiana, and Texas; the Territories of Oklahoma and Indian Territory; the Republic of Mexico; the West India Islands; and all countries in Central America and South America.

District No. 7.—The States of North Dakota, South Dakota, Nebraska, Kansas, Montana, Wyoming, Colorado, Idaho, Utah, Washington, Oregon, California, and Nevada; the following Territories: New Mexico, Arizona, Hawaii; and all countries in Asia and Australasia.


AMERICAN SOCIETY OF CIVIL ENGINEERS.
MAP SHOWING THE
SEVEN GEOGRAPHICAL DISTRICTS INTO WHICH THE TERRITORY OCCUPIED BY THE MEMBERSHIP IS DIVIDED
No. 1. Territory within 50 miles of New York P.O.
No. 2 Includes Europe and Africa
No. 3 Includes Alaska.
No. 6 Includes Mexico, Central America, South America and the West India Islands
No. 7 Includes Hawaii, Asia and Australasia
1
2
3
4
5
6
7
PACIFIC OCEAN
ATLANTIC OCEAN
GULF OF MEXICO
DOMINION OF CANADA
MEXICO
LOWER CALIFORNIA
CUBA
HAITI
WASHINGTON
OREGON
CALIFORNIA
NEVADA
IDAHO
MONTANA
UTAH
ARIZONA
WYOMING
COLORADO
NEW MEXICO
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NEW YORK
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MASS.
N. J.
Columbia
R.
Colorado
Rio Grande
Missouri
Mississippi
Ohio
St. Lawrence R.
120°
110°
100°
90°
80°
70°
60°
130°
40°
30°
20°

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all meetings:

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Society of Engineers, 17 Victoria Street, Westminster, S. W., England.

American Institute of Mining Engineers, 99 John Street, New York City.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Civil Engineers' Club of Cleveland, 1200 Scofield Building, Cleveland, Ohio.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Philadelphia, 1122 Girard Street, Philadelphia, Pa.

Engineers' Society of Western Pennsylvania, 410 Penn Avenue, Pittsburg, Pa.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Engineers' Club of Central Pennsylvania, Corner, Second and Walnut Streets, Harrisburg, Pa.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Teknisk Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Société des Ingénieurs Civils de France, 19 Rue Blanche, Paris, France.

Svenska Teknologförenigen, Brunkebergstorg 18, Stockholm, Sweden.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Sachsischer Ingenieur- und Architekten- Verein, Dresden, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Pacific Northwest Society of Engineers, 617–618 Pioneer Building, Seattle, Wash.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Memphis Engineering Society, Memphis, Tenn.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

The Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Cleveland Institute of Engineers, Middlesbrough, England.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

SEARCHES IN THE LIBRARY.

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling, compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

Copies of all lists of references are filed, so that in many cases it is only necessary to make a typewritten copy, which reduces the cost of searches to a minimum.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

ACCESSIONS TO THE LIBRARY.

From March 14th to April 7th, 1906.

DONATIONS.*

A TREATISE ON PRODUCER-GAS AND GAS-PRODUCERS.

By Samuel S. Wyer. Cloth, 9 x 6 in., illus., 296 pp. New York, The Engineering and Mining Journal, 1906. $4.

The preface states that the first four chapters of the book are for the benefit of readers who may not be familiar with those fundamental laws and definitions of physics and applied chemistry upon which a rational discussion of producer-gas must be based. Since the engineering side of gas-producers is so closely related to applied chemistry, temperatures are stated either in Centigrade or Fahrenheit; however, as the book is intended primarily for engineers, the Fahrenheit scale is used mostly. The book contains a bibliography of gas-producers of fourteen pages, arranged chronologically. There is an index of five and one-half pages. The Contents are: Fundamental Physical Laws and Definitions; Fundamental Chemical Laws and Definitions; Thermal and Physical Calculations; Commercial Gases; Status of Producer-Gas; Classification of Gas-Producers; Manufacture and Use of Producer-Gas; Use of Steam in Gas-Producers; Carbon Dioxide in Producer-Gas; Efficiency of Gas-Producers; Heat Balance of the Gas-Producer; Fuel; Requirements; History of Gas-Producers; American Pressure Producers; American Suction Gas-Producers; Gas-Cleaning; By-Product Gas-Producers; By-Product Coke Oven Gas-Producers; Producer-Gas for Firing Ceramic Kilns; Producer-Gas for Firing Steam Boilers; Wood Gas-Producers; Removal of Tar from Gas; Gas-Producer Power Plants; Operation of Gas-Producers; Testing Gas-Producers; Future of the Gas-Producer; Gas-Poisoning; Reference Data; Bibliography.

MÉTHODES ÉCONOMIQUES DE COMBUSTION DANS LES CHAUDIERES A VAPEUR.

Par J. Izart. Paper, 10 x 6 in., illus., 15 + 213 pp. Paris, H. Dunod et E. Pinat, 1906. 7 francs 50 centimes.

It is stated that this work continues the author's series of studies on industrial economy, and is meant to indicate clearly and simply the methods to be followed in order to realize an economy of fuel. The second part of the volume contains tables and formulas which are said to be valuable to the engineer. There are five general divisions in the contents of the book: Etude Economique de la combustion; Pertes et rendement dans la combustion; Choix d'un combustible Economique; Economie dans les méthodes de chauffe; Appareils pour le contrôle de la chauffe. There is an alphabetical index of five pages.

A PRACTICAL TREATISE ON FOUNDATIONS;

Explaining Fully the Principles Involved, Supplemented by Articles on the Use of Concrete in Foundations. By W. M. Patton. Second Edition, Enlarged. Cloth, 9 x 6 in., illus., 28 + 549 pp. New York, John Wiley & Sons, 1906. $5.

The preface states that this edition has been enlarged by the addition of one hundred and thirty-five pages. The extensive use of concrete for foundations receives special notice and alone would justify the addition. Numerous descriptions of important modern structures are given in sufficient detail to make them understood. The index to the addition has been incorporated with the old index, and the whole is revised so that the numbers now refer to pages instead of to articles and paragraphs, as before. The index covers fourteen pages.

THE OYSTER;

A Popular Summary of a Scientific Study. By William K. Brooks. Second and Revised Edition. Cloth, 8 x 6 in., illus., 14 + 225 pp. Baltimore, The Johns Hopkins Press, 1905. $1.

*Unless otherwise specified, books in this list have been donated by the publisher.

It is stated in the preface that this book is written for the information of all who care for oysters,—no matter whether their point of view be that of providers or consumers,—of the oysterman, the money-maker, the housekeeper, the legislator, the editor, or the student of natural history. It is the purpose of the author to help to bring about a practical and judicious system of oyster farming in Maryland, and the development and improvement of the natural resources of the waters by an account of the way in which the structure and habits of the oyster fit it for cultivation, as a submarine agricultural product. It is stated that, in this edition, no essential change seems to be necessary, and most of the new matter refers to minor points, with one exception. There has been added to the account of the structure of the oyster a section upon its peculiar fitness for gathering up the germs of cholera and typhoid fever and transmitting them to man, since, it is stated, the importance of clear ideas upon this subject increases with the growth of the cities and towns upon tidal shores, and with the increasing danger of the pollution of the oyster area by sewage. The book contains a bibliography, but there is no index.

MAN AND THE EARTH.

By Nathaniel Southgate Shaler. Cloth, 8 x 5 in., 6 + 240 pp. New York, Fox, Duffield and Company, 1905. $1.50 net.

The author has endeavored to set forth certain reasons why there should be a change in the point of view from which we commonly regard the resources of the earth, with the hope of directing attention to the future of the material values of the earth. The author notes that the statements concerning the mineral and other material resources are not supported by statistics in these pages, and, although the stores of value to men can be estimated in general terms, there is, as yet, no sufficient basis for accurate quantitative reckonings. The Contents are: Earth and Man; The Future of Power; The Exhaustion of the Metals; The Unwon Lands; Land from the Waters; The Problem of the Nile; The Maintenance of the Soil; The Resources of the Sea; The Changes to Come in the Human Period; The Beauty of the Earth; The Future of Nature upon the Earth; The Last of Earth and Man; The Attitude of Man to the Earth—Summary and Conclusions. There is an index of five and one-half pages.

A HANDBOOK ON REINFORCED CONCRETE;

For Architects, Engineers and Contractors. By F. D. Warren. Cloth, 7 x 4 in., illus., 271 pp. New York, D. Van Nostrand Company, 1906. $2.50.

The preface states that the author has endeavored to produce a reference handbook in preference to a textbook. It was the purpose to have a work treating of a general form of design rather than of a particular or patented system, but to which any of the latter may be applied. The treatment of the many phases entering the design has been carried out along well-known formulas based upon the theory of elasticity, but modified by the usual assumptions, such as the "conservation of planes" and "Hookes' Law," and not upon empirical formulas based upon experiments. Attention should be called to the fact that before applying the theory of elasticity to any particular part of the design, a sufficient number of tests were carried out along this basis to approve it, and determine the coefficients and constants. The book is divided into four parts: Part I gives a general but concise résumé of the subject from a practical standpoint, bringing out some of the difficulties met with in practice, and suggesting remedies. Under Part II is compiled a series of tests. Part III contains tables from which it is hoped the designer may obtain all necessary information to meet the more common cases in practice. Part IV treats of the design of trussed roofs from a practical standpoint. There is a table of contents, but no index.

Gifts have also been received from the following:

Alaska Mexican Gold Min. Co. 2 pam.
Alaska United Gold Min. Co. 1 pam.
Am. Gas Light Assoc. 1 bound vol.
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Ginn & Co. 1 pam.
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North, E. P. 12 pam., 2 bound vol., 7 vol.
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U. S. Interstate Commerce Comm. 3 bound vol., 3 pam.
U. S. Lake Surv. Office. 1 map.
U. S. War Dept. 4 bound vol., 14 specif.
Waddell, J. A. L. 1 pam.
Winnipeg, Man.—City Engr. 2 pam.
Wis.—Geol. and Natural History Surv. 2 bound vol.

BY PURCHASE.

Lippincott's New Gazetteer; a Complete Pronouncing Gazetteer or Geographical Dictionary of the World. Edited by Angelo and Louis Heilprin. Philadelphia and London, J. B. Lippincott Company, 1906.

Report of the Royal Commission on London Traffic, Vols. V-VI. London, Wyman and Sons, Limited, 1906.

The Adjustment of Observations by the Method of Least Squares with Applications to Geodetic Work. By Thomas Wallace Wright and John Fillmore Hayford, Assoc. M. Am. Soc. C. E. Second Edition. New York, D. Van Nostrand Company, 1906.

Coal-Tar and Ammonia. By George Lunge. Third and Enlarged Edition. London, Gurney and Jackson, 1900.

Building Construction and Superintendence; Part III, Trussed Roofs and Roof Trusses. By F. E. Kidder. New York, William T. Comstock, 1906.

The Biographical Directory of the Railway Officials of America, 1906. Edited and Compiled by T. A. Busbey. Chicago, Railway Age Company, 1906.

SUMMARY OF ACCESSIONS.

March 14th to April 7th, 1906.

Donations (including 21 duplicates)	208
By purchase	7
Total	215

MEMBERSHIP.

ADDITIONS.

MEMBERS.

		Date of Membership.
ANDERSON, GEORGE GRAY. Cons. Engr., 501 Equitable Bldg., Denver, Colo		Feb. 7, 1906
ARANGO, RICARDO MANUEL. Div. Engr., Div. of Meteorology and River Hydraulics, Isthmian Canal Comm., Panama, Panama.	Assoc. M. M.	Sept. 2, 1896 Feb. 6, 1906
BAUM, GEORGE. Supt., The Niagara Constr. Co., Ltd., Niagara Falls, Ont., Canada..	Jun. Assoc. M. M.	Jan. 4, 1892 April 5, 1899 Mar. 6, 1906
COOMBS, PHILIP HENRY. Civ. Engr. and Surv., 12 Exchange Blk., Bangor, Me		Mar. 7, 1906
DABNEY, THOMAS GREGORY. Chf. Engr., Yazoo-Mississippi Delta Levee Dist., Clarksdale, Miss		Jan. 3, 1906
DAVIS, ARTHUR LINCOLN. Asst. to Pres., Am. Bridge Co. of N. Y., 42 Broadway, New York City	Jun. Assoc. M. M.	April 4, 1893 Oct. 7, 1896 Mar. 6, 1906
DILL, HOWARD ALBERT. Engr. and Supt., Richmond City Water Works, Richmond, Ind		Oct. 4, 1905
GARLINGHOUSE, FREDERICK LEMAN. Glenshaw, Pa		Mar. 7, 1906
KENNEY, EDWARD FULBISTER. Engr. of Tests, P. R. R. Co., Broad St. Station, Philadelphia, Pa	Jun. Assoc. M. M.	Nov. 3, 1896 May 3. 1899 April 3, 1906
LEWERENZ, ALFRED COURTNEY. Civ. Engr., U. S. N., U. S. Navy Yard, Puget Sound, Bremerton, Wash	Jun. Assoc. M. M.	Mar. 5, 1895 June 6, 1900 Mar. 6, 1906
MURRAY, JOHN FRANCIS. 25 South 34th St., Philadelphia, Pa		Mar. 7, 1906
NORBOE, PAUL MANINGHAM. Hanford, Cal		Nov. 1, 1905
PROAL, ARTHUR BREESE, JR. 2643 Broadway, New York City		Mar. 7, 1906
SILVA FREIRE, VICTOR DA. P. O. Box 18, Sao Paulo, Brazil		Jan. 3, 1906
SKINNER, JOHN FRANKLIN. 23 Cornell St., Rochester, N. Y		April 4, 1906
SPOONER, HERMAN WINSLOW. 28 Granite St., Gloucester, Mass		Jan. 3, 1906
SWEZEY, EDWIN CHARLES. 259 Fifty-second St., Brooklyn, N. Y		Mar. 7, 1906
WILKES, JAMES KNAPP. Chf. Engr. of Sewers, City Hall, New Rochelle, N. Y	Assoc. M. M.	June 3, 1891 April 3, 1906
WILLIAMS, WILLIAM FISH. City Engr., S. E. cor. Court and Orchard Sts., New Bedford, Mass		April 4, 1906

MEMBERS (*Continued*).

		Date of Membership.
WOOD, IRVING SPARROW. Asst. Engr. in Chg., Water Dept., City Engr.'s Office, City Hall, Providence, R. I.	Jun. Assoc. M. M.	Mar. 5, 1890 Mar. 7, 1900 Mar. 6, 1906

ASSOCIATE MEMBERS.

AIKENHEAD, JAMES RAY. Asst. Engr., Westinghouse, Church, Kerr & Co., East Liverpool, Ohio		Feb. 7, 1906
ALEXANDER, HENRY JAMES. Asst. Engr., Rapid Transit Comm., 231 West 125th St., New York City		Mar. 7, 1906
BUEHLER, WALTER. 400 Oneida Blk., Minneapolis, Minn.		Mar. 7, 1906
CLASS, CHARLES FRANK. Asst. Chf. Draftsman, Am. Coke & Gas Constr. Co. (Res., 1440 Kenwood Ave.), Camden, N. J.	Jun. Assoc. M.	Mar. 3, 1903 April 4, 1906
ESTES, FRANKLIN EDWARD. Rincon Antonio, Oaxaca, Mexico		Feb. 7, 1906
FALTER, PHILIP HENRY. Shawinigan Falls, Que., Canada.		Mar. 7, 1906
GOODELL, JOHN STANTON. Asst. Engr., G. C. & S. F. Ry., P. O. Box 472, Cleburne, Tex.		Mar. 7, 1906
HOLTZMAN, STEPHEN FORD. 11 East 24th St., New York City		Mar. 7, 1906
HOWELL, FRANK SCOTT. 2394 Seventh Ave., New York City		April 4, 1906
KEITH, CHARLES WHITESIDE. 209 Adams St., Room 62, Chicago, Ill.		Mar. 7, 1906
LEANE, WALTER BURDITT. Res. Engr., Fish River Irrigation Surveys, Fish River Station, Cape Colony, South Africa		Jan. 3, 1906
LEE, ELSWORTH MORTIMER. Care, Miller-Collins Co., 29 Broadway, New York City		Mar. 7, 1906
LEIGHTON, MARSHALL ORA. 1330 F St., Washington, D. C.		Oct. 4, 1905
LINDAU, ALFRED EMANUEL. Asst. Engr., St. Louis Expanded Metal Fireproofing Co., 606 Century Bldg., St. Louis, Mo.		Jan. 3, 1906
MUNN, ALEXANDER MAJORS. Nebraska City, Nebr.		Jan. 3, 1906
PARKER, RICHARD DENNY. Res. Engr., Houston, East & West Tex. Ry., and Houston & Shreveport R. R., Houston, Tex.	Jun. Assoc. M.	June 3, 1902 Mar. 7, 1906
ROBINSON, GEORGE LOOMIS. 1 Madison Ave., New York City	Jun. Assoc. M.	April 5, 1904 Mar. 7, 1906
STOKES, STANLEY FREDERICK. Govt. Engr., Mines Dept., Johannesburg, Transvaal, South Africa		Oct. 4, 1905
SUTTON, CHARLES WOOD. 337 Broad St., Chester, Pa.	Jun. Assoc. M.	Dec. 1, 1903 Dec. 6, 1905
THAYER, HORACE RICHMOND. 680 Ostrum St., South Bethlehem, Pa.		Mar. 7, 1906

ASSOCIATE MEMBERS (*Continued*).

		Date of Membership.
WILLIAMS, ROGER BUTLER, JR. Pres., Williams & Whitman, Inc., 17 Battery Pl., New York City	Jun. Assoc. M.	Oct. 1, 1901 Mar. 7, 1906

ASSOCIATES.

AMBLER, DANIEL GRIFFITH. Hotel Hamilton, Washington, D. C	Mar. 7, 1906
MERIWETHER, COLEMAN. Pres. and Gen. Mgr., Lock Joint Pipe Co., 346 Broadway, New York City	Feb. 7, 1906

JUNIORS.

BLATCH, NORA STANTON. 17 East 26th St., New York City	Mar. 6, 1906
BONIFACE, ARTHUR. 605 East 139th St., New York City	Jan. 2, 1906
BREMER, LEWIS PAUL. Webster Ave., New Rochelle, N. Y.	Feb. 6, 1906
BROWN, ARTHUR ROBERT. Care, F. B. Maltby, Cristobal, Canal Zone, Panama	Jan. 2, 1906
BUSHNELL, HOWARD EMORY. 115 Elm St., Hartford, Conn	Mar. 6, 1906
CADWALADER, GOUVERNEUR. Perry Bldg., Philadelphia, Pa	April 3, 1906
FORD, HARRY CLIFFORD. Box 131, Portsmouth, Va	Mar. 6, 1906
FRAZER, JAMES STANLEY. 56 Clark St., Brooklyn, N. Y.	Jan. 2, 1906
FRENCH, ROGER DELAND. 432 N. Center St., Terre Haute, Ind	Mar. 6, 1906
JONES, PUSEY. Care, N. Y. W. & B. Ry., 30 Broad St., New York City	Mar. 6, 1906
KING, CLIFFORD MARSHALL. Asst. Engr., U. S. Reclamation Service, Rupert, Idaho	Mar. 6, 1906
MCNEIL, ARTHUR JAMES. Box 462, Whittier, Los Angeles Co., Cal	Mar. 6, 1906
PIKE, RALPH ASHUR. 145 N. High St., Mt. Vernon, N. Y.	Mar. 6, 1906
REYNOLDS, LAFAYETTE CLOWE. 311 North Seventh Ave., Mt. Vernon, N. Y	Feb. 6, 1906
STEEGMULLER, CHARLES ALBERT AUGUSTINE. 171 Eleventh St., Long Island City, N. Y	Mar. 6, 1906
STILES, ALBERT IRVINE. Cuerpo de Ingenieros de Minas, Apartado 949, Lima, Peru	Feb. 6, 1906
WATSON, GEORGE LINTON. 54 Baxter Bldg., Philadelphia, Pa.	Mar. 6, 1906

DEATHS.

HAUSMAN, FREDERICK APPEL. Elected Junior, February 3d, 1903; died March 6th, 1906.

HEMMING, DUNKIN WIRGMAN. Elected Associate Member, September 7th, 1892; Member, May 31st, 1904; died March 22d, 1906.

PARKHURST, HENRY WILLIAMS. Elected Member, September 5th, 1877; died April 7th, 1906.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(March 11th to April 7th, 1906.)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Street Railway Journal*, New York City. Issues for first Saturday of each month 20c., other issues 10c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England, 25c.
(22) *Iron and Coal Trades Review*, London, England, 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Engineer*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, $1.
(29) *Journal*, Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l'Assoc. des Ing. Sortis des Ecoles Speciales de Gand*, Brussels, Belgium.
(32) *Memoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Genie Civil*, Paris, France.
(34) *Portefeuille Economique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Revue de Mecanique*, Paris, France.
(38) *Revue Generale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill., 10c.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussees*, Paris, France.
(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 8c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.

(54) *Transactions*, Am. Soc. C. E., New York City, $5.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $5.
(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Eng. Soc. W. Pa., 410 Penn Ave., Pittsburg, Pa., 50c.
(59) *Transactions*, Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E , London, England.
(64) *Power*, New York City, 20c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 15c.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England.
(70) *Engineering Review*, New York City, 10c.
(71) *Journal*, Iron and Steel Inst., London, England.
(72) *Street Railway Review*, Chicago, 30c.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England.
(78) *Beton und Eisen*, Vienna, Austria.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie-Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(82) *Dinglers Polytechnisches Journal*, Berlin, Germany.
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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

DISPOSAL OF MUNICIPAL REFUSE, AND RUBBISH INCINERATION.

By H. de B. Parsons, M. Am. Soc. C. E.

To be Presented June 6th, 1906.

During the winter of 1902-03, the writer contributed to the informal discussion on "The Sanitary Disposal of Municipal Refuse."* At that time, some data were presented and a short description was given of the rubbish incinerator, built from the writer's plans, at the foot of West Forty-seventh Street, New York City. Subsequently, additional data were presented at the International Engineering Congress at St. Louis.†

The writer now contributes to the Society further data on the subject of municipal refuse, which he hopes will prove both interesting and instructive, and a description of the rubbish incinerating plant built on Delancey Slip, Borough of Manhattan, City of New York, by order of John McGaw Woodbury, Assoc. Am. Soc. C. E., Commissioner of the Department of Street Cleaning, together with a description of the adjoining electric lighting station (built by order of the Department of Bridges), which utilizes the heat pro-

* *Transactions*, Am. Soc. C. E., Vol. L, p 95.

† *Transactions*, Am. Soc. C. E.. Vol. LIV. Part E, p. 263.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

duced from the incineration of the rubbish to light the Williamsburg Bridge.

This combined plant was built by the Departments of Street Cleaning and of Bridges for the benefit of the municipality, and each department retained the writer as Consulting Engineer to prepare the plans and superintend the construction. The Department of Street Cleaning gains by having the rubbish collections reduced in bulk and transformed into ash which is valuable for land-fills, and by the improved sanitary conditions caused by completely burning the beds, bedding, and old furniture, which are often germ laden, and thus preventing them from ever returning to the city. The Department of Bridges saves in the cost of lighting the bridge by using the heat of combustion for the generation of electricity. To show how well these objects have been realized is part of the subject of this paper.

General Data.

Portions of the data here given have been taken from the papers mentioned previously. The repetition has been made in order that the new data presented, together with the former data, may be brought into this contribution, and the whole rendered as complete as possible.

A general division and subdivision of the materials is given in Table 1. The scheme classifies the wastes of a municipality and places them in three divisions, the basis of classification being chiefly dependent upon the method of handling, as practiced by "usage and custom." The first and second divisions—the "fluid and semi-fluid refuse" and the "general refuse"—consist of that portion for which the authorities should make provision, as the people cannot be made, or entrusted, to dispose of them in a sanitary manner. The third division includes those wastes which are cared for by private or special service—either by private contract or by the parties creating the refuse. The item of dead animals, however, is a possible exception, and, in large cities, the authorities should make some provision for their removal.

As uniformity in practice is lacking, the scheme for disposal adopted in any city will not always agree with the method of division for classification as given in Table 1. Some cities provide for the disposal of only a part of the general refuse, while others care for the whole.

The general refuse, which is the subject of this paper, and for which the community should provide a method for collection and disposal, is separated into five classes:

I.—Ashes;
II.—Garbage;
III.—Rubbish;
IV.—Street-sweepings;
V.—Snow.

TABLE 1.—Classification of City Wastes.

City Waste Materials.........	Fluid and Semi-fluid Refuse.	Sewage....	House Sewage.
			Street, Roof and Area Drainage.
			Night Soil.
	General Refuse...	Ashes......	Steam Ashes.
			Household Ashes.
		Garbage...	Animal Matter.
			Vegetable Matter.
			Meat and Bones.
			Fruit.
		Rubbish...	Paper.
			Wood.
			Rags and Bedding.
			Leather and Rubber.
			Metals.
			Bottles, Glass and Crockery.
			Sweepings from Buildings.
		Street-Sweepings.	Animal Manure.
			Pavement Dirt.
			Droppings from Carts.
			Materials from Building Construction.
			Some Rubbish and Leaves.
		Snow.	
	Trade Refuse............		Cellar Excavations.
			Materials from Building Construction.
			Stable Manure.
			Market Offal.
			Slaughter-house Offal.
			Dead Animals.

The characteristics of the general refuse divisions can be stated as follows:

1.—Ashes.—Ashes consist of silica, oxide of iron, potash, alumina, lime, magnesia, soda, barium, phosphorus in phosphates, sulphur in sulphates, etc., and of unburned coal.

Ashes weigh about 1 350 lb. per cu. yd., varying from 1 200 to 1 500 lb., and can be divided into two grades or classes, *viz.:* "steam-ashes" and "household-ashes."

The steam-ash is that which is obtained from coal burned under steam boilers, in those industrial works where care is taken to have

FIG. 1.—REFUSE DUMPED DIRECTLY INTO SCOWS.

FIG. 2.—FRONT OF INCINERATOR BUILDING AT DELANCEY SLIP.

FIG. 3.—MATERIAL PICKED OUT, BALED AND READY FOR SHIPMENT.

the combustion reasonably perfect. The amount of combustible in steam-ash varies from about 11 to 50%,* increasing as the size of the coal decreases and as the coal is of a less caking character. This amount of combustible in the steam-ash represents a loss of unburned coal in the ash of from 2 to 33% of the fuel. A fair general average could be assumed at 26% by weight of the ash, or at 4% of the total coal used. With mechanical stoking grates, for the small sizes of anthracites and for the anthracites of a friable nature, the amount of combustible in the ash would probably exceed the average just stated. For the ordinary anthracites, the average would approximate 30%; and for the bituminous coals, the average would be about 24 per cent. For the caking coals, the average would be excessive. The loss would also be greater than the foregoing average for coals containing large percentages of earthy matter.

Table 2 shows the percentage of combustible matter found in the ash, as reported in a number of steam boiler trials, being the percentage by difference between the ash as determined by analysis and that as recovered from the ash-pit and weighed.

The household-ash is that which is obtained from coal burned in house-heaters, ranges, stoves and open fire-places. The coal used is almost entirely of those sizes which will pass through a 4½-in. screen and over a 1¼-in. screen, with the exception of the cannel coal consumed in open fire-places. While this cannel naturally burns to a clean ash, still a large quantity falls through the basket grates; and, furthermore, as the coal is of such a soft nature, much of it, in handling, crumbles into dust and small pieces, and is thus rendered too small for household use. All this combustible portion finds its way into the ash collections. On the continent of Europe, a smaller proportion of unburned coal is found in the household-ash collections than in the United States, as the use of closed stoves for household purposes is so common. In England, the proportion is greater, on account of the numerous open fire-places.

For the past twelve years, the writer has kept a complete coal record for his private dwelling. The cannel coal dust which collected between August, 1896, and August, 1902, and which had to be discarded from the household, amounted to 8.3% of the weight of cannel coal purchased. Assuming that this cannel coal dust con-

*See "Steam Boilers," by H. de B. Parsons, p. 23.

tained 4% of true ash, then the combustible portion discarded was about 8% in addition to that which was contained in the ashes from the grates.

TABLE 2.—Unburned Coal in Steam-Ash.

Kind of coal.	Percentage of Ash.		Percentage of Combustible.	
	By analysis.	By boiler trial.	Based on ash.	Based on coal.
	Per cent.	Per cent.	Per cent.	Per cent.
* Cumberland Bituminous	5.0	7.5	33.3	2.5
* Pocahontas "	4.0	10.0	60.0	6.0
* Cumberland "	6.1	8.1	24.7	2.0
* Clearfield "	7.6	10.1	24.7	2.5
* Cape Breton "	7.8	9.4	17.0	1.6
† George's Creek "	9.0	10.6	15.1	1.6
† W. V. Pocahontas "	11.1	11.9	6.7	0.8
† George's Creek "	9.3	10.5	11.4	1.2
† Buckwheat, No. 1, Anthracite	17.5	22.5	22.2	5.0
† Buckwheat " "	16.2	24.9	35.0	8.7
* Chestnut, No. 2, "	12.0	16.0	25.0	4.0
* Chestnut " "	11.8	19.3	38.8	7.5

General average........36 per cent.
Average, Anthracite........30 "
" Bituminous........24 "

* Barrus on "Boiler Tests," page 251.
† From tests made by D. S. Jacobus.

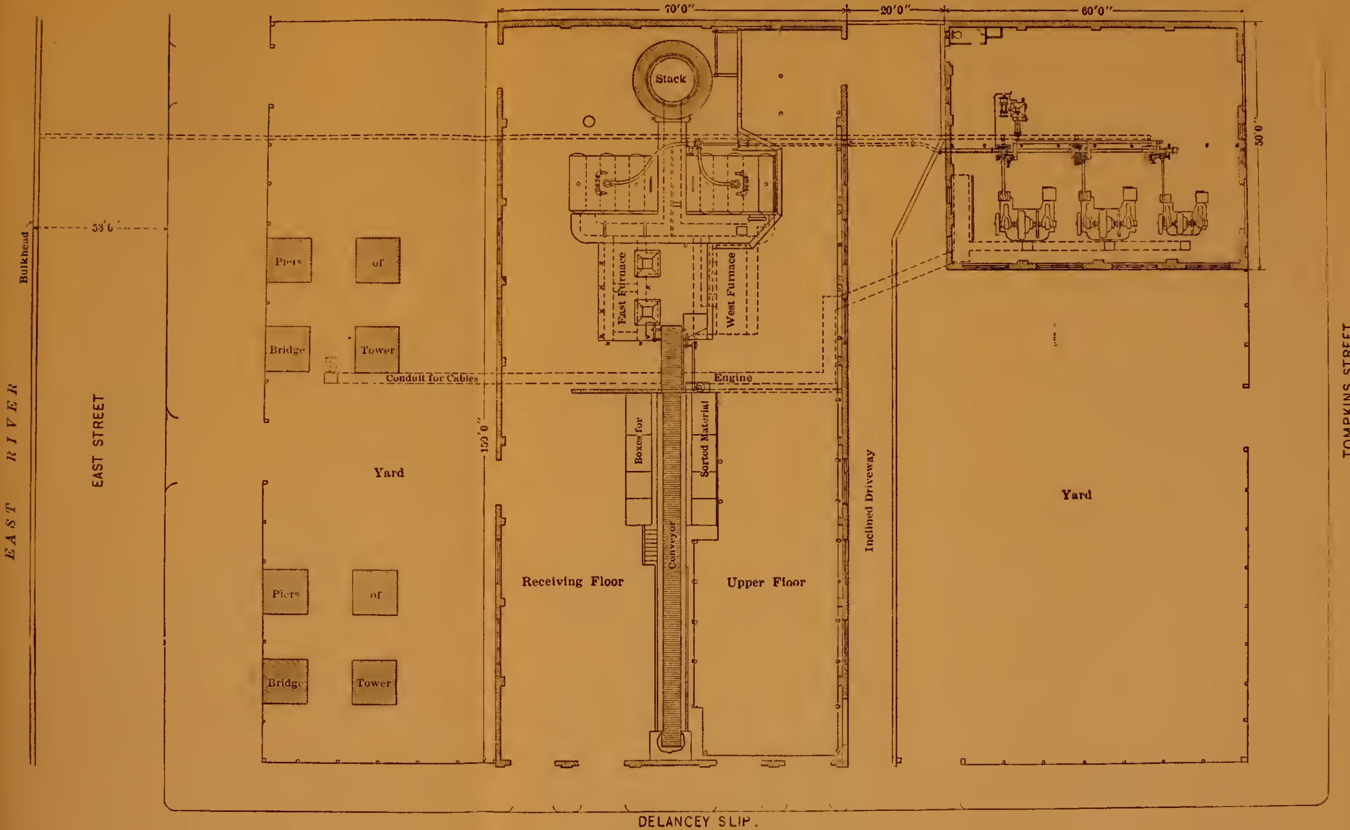
The proportion of cannel to anthracite used was 1 to 7.6 or 13 per cent. By analysis, the combustible matter in the cannel coal ashes was 21.8%, making a total combustible of about 29.8 per cent. Also, by analysis, the combustible matter in the anthracite ash was 13.4 per cent. Therefore, the real average combustible in the ash discarded from the house, which can be taken as an average of similar houses in the residential portion of New York City, amounted to 15.3 per cent.

The writer had careful analyses made in February and March, 1904, of some household ashes and of samples of ash obtained directly from some of the dumps of the City of New York, with the results shown in Table 3. Therefore, it would seem safe to state that in a city like New York, the mixed ash collections from all quarters contain from 30 to 35% of combustible matter.

All the analyses of ash were made by Messrs. Simonds and Wain-

PLATE XXXI.
PAPERS, AM. SOC. C. E.
APRIL, 1906.
PARSONS ON
REFUSE DISPOSAL.

RUBBISH INCINERATOR AND ELECTRIC LIGHTING STATION, GENERAL ARRANGEMENT OF PLANT.



wright, Analytical Chemists. The samples from the city dumps were made by taking a shovelful from alternate carts as they drove on the dumps during the major part of one day at a dump. The pile was mixed and quartered, and the final sample, of from 12 to 15 lb., was crushed in a mill and again quartered.

TABLE 3.—ANALYSES OF ASHES.*

	Moisture at 102° C.	Pure ash.	Combustible matter.	Total.
HOUSEHOLD ASH.	Per cent.	Per cent.	Per cent.	Per cent.
Coal burned in a stove, anthracite, size "stove No. 2"	0.36	90 81	8.83	100.00
Coal burned in a hot-air furnace, anthracite, size "egg"	0.06	86.50	13.44	100.00
Coal burned in an open grate, English cannel, size "6-in. cubes"	0.64	77.53	21.83	100.00
MIXED ASHES.				
Samples from New York City Dumps...				
Clinton Street	1.69	62.19	36.12	100.00
Stanton Street	0.80	67.43	31.77	100.00
West Forty-seventh Street	0.83	63.73	35.44	100.00
Average	1.11	64.45	34.44	100.00

* Made for the writer by Messrs. Simonds and Wainwright.

II.—*Garbage.*—Garbage is, by far, the most important division of general refuse, because it is the most difficult to handle without causing annoyance, is unsightly, and is likely to become putrescible and diffuse offensive odors. On the other hand, it has a distinct commercial value, due to the value of the by-products which may be obtained from it. Whether this value is worth saving, in practice, is still a moot question, and no unity of sentiment exists, although much has been written concerning its treatment.

Its composition differs according to the season of the year, as well as the location of the city and the character of the district from which it is collected. Garbage contains a large proportion of water, the amount varying from about 50% to more than 80 per cent. During the fruit and green vegetable seasons the amount of water is always large, while in the ordinary dwelling-house garbage in winter the amount is nearer the lower limit.

In Table 4 are given some analyses of the composition of garbage. Analysis *A* was made by the Sanitary Bureau of the Board of Health of the City of New York, in October, 1897. The sample

was taken from the garbage as received on the barges or scows for final removal, and may be considered an average of the collections from many and various sources. It was practically pure garbage.

TABLE 4.—Analyses of Garbage.

	New York City. A.	Brooklyn, N. Y. B.	Trenton, N. J. C.	United States. D.	England. E.	Berlin. F.
	Per cent.	Per cent.	Per cent.	Per cent.	Per cent.	Per cent.
Water	65.90	71.00	80.00	70.00	65.00	60.00
Animal and vegetable solids	25.62	20.00	16.80	20.00	24.00	30.00
Grease	....	2.00	....	3.00	2.00	2.00
Paper	....	7.00	2.40	7.00	9.00	8.00
Rags, etc	....		0.60			
Boxes	....		0.20			
Mineral matter	8.48	....	....	....	....	....
Totals	100.00	100.00	100.00	100.00	100.00	100.00
Fat	7.07	....	....	....	....	....
Total nitrogen	0.86	....	....	....	....	....
Phosphoric acid, P_2O_5	0.07	....	....	....	....	....
Potash, K_2O	0.30	....	....	....	....	....

A. Dept. of Health, October 15th, 1897.
B. Report, Brooklyn Board of Health. 1896.
C. *Transactions*, Am. Soc. C. E., Vol. L, p. 128.
D, *E* and *F*. International Engineering Congress, St. Louis, *Transactions*, Am. Soc. C. E., Vol. LIV, Part E, p. 263.

Analysis *B* is an analysis of Brooklyn summer garbage, as stated in the report of Joseph B. Taylor.*

Analysis *C* was made by Theodore Horton, Assoc. M. Am. Soc. C. E., of garbage as collected in Trenton, N. J., during the summer of 1902. It was obtained by sorting ½-ton samples and weighing and averaging.

Analysis *D* gives the composition of average American garbage, as stated by MacDonough Craven.

Analyses *E* and *F* are quoted on the authority of Rudolph Hering, who obtained his figures from W. F. Goodrich, of London, and Messrs. Bohm and Grohn, of Berlin.

All the figures in the table are given in percentages, and do not indicate more than the relative proportions of the elements reported. The actual quantities collected in certain cities are stated later.

* Board of Health Report, Brooklyn, 1896.

FIG 1.—BELT-CONVEYOR AND SORTING-BOXES.

FIG. 2.—END OF BELT-CONVEYOR AT PLATFORM OVER FURNACES.

Garbage, as ordinarily collected, contains some rubbish, such as paper, cans, boxes, etc. Even in those cities where a primary separation system is in vogue, this admixture of rubbish may amount to 5 or 6% or more by weight.

Pure garbage weighs about 1 100 or 1 200 lb. per cu. yd., when loosely collected.

III.—Rubbish.—Rubbish is discarded trash, principally, of all kinds of paper, wood, rags, mattresses, bedding, boxes, chairs, sofas, barrels, leather, old shoes, rubber, tin cans, metal scraps, bottles, broken glass, crockery and the like. It is a most heterogeneous aggregation, and contains all the household wastes that cannot be classified as ashes or garbage. Every conceivable kind of rubbish waste is discarded from the houses or found in the street-sweepings; but the distinction between these classes of refuse is that such of the material as is collected from the houses strictly belongs to the rubbish classification, and such as is obtained from the streets to the street-sweeping classification.

Rubbish, as ordinarily piled in the carts, or without extra packing, weighs from 130 to 225 lb. per cu. yd. In Boston, Mass., the average weight, as delivered at the Atlantic Avenue collecting and incinerating station, was 202 lb. per cu. yd. In New York the average weight at the Thirtieth Street dump was 143 lb. per cu. yd.; at the Forty-seventh Street dump, 141.1 lb., and at the Delancey Slip incinerator, 139 lb.

As rubbish contains the dirt and dust from sweepings, cast-away bedding and rags, it is likely to harbor germs of disease, and should be taken to the place for final disposition as directly as possible, so that dry, germ-bearing dust may not be scattered among the people of the community. In other respects, the rubbish collections may be termed clean, and are not especially disagreeable to handle.

Some analyses of rubbish are given in Table 5. The figures in the first column are measurements made by the writer in 1905, and in the second and third columns by F. L. Stearns, Assoc. M. Am. Soc. C. E., of the Department of Street Cleaning of New York City, in 1904. These figures, together with those for Boston, are the percentages by weight of the marketable portions only, that is, that portion which was picked out and sold. The remainder was valueless, and was not classified, but was composed of the same ele-

ments as mentioned, together with bedding, mattresses, furniture, etc.

The smell arising from a collection of rubbish is not offensive, and the mass does not decompose. At times the collections contain some garbage, which on decomposing gives the mass a decided smell.

Rubbish, like garbage, has an inherent value. A considerable portion of the mass can be sorted out and sold at a profit. In consequence, the collections are often picked over, for which privilege contractors can be found.

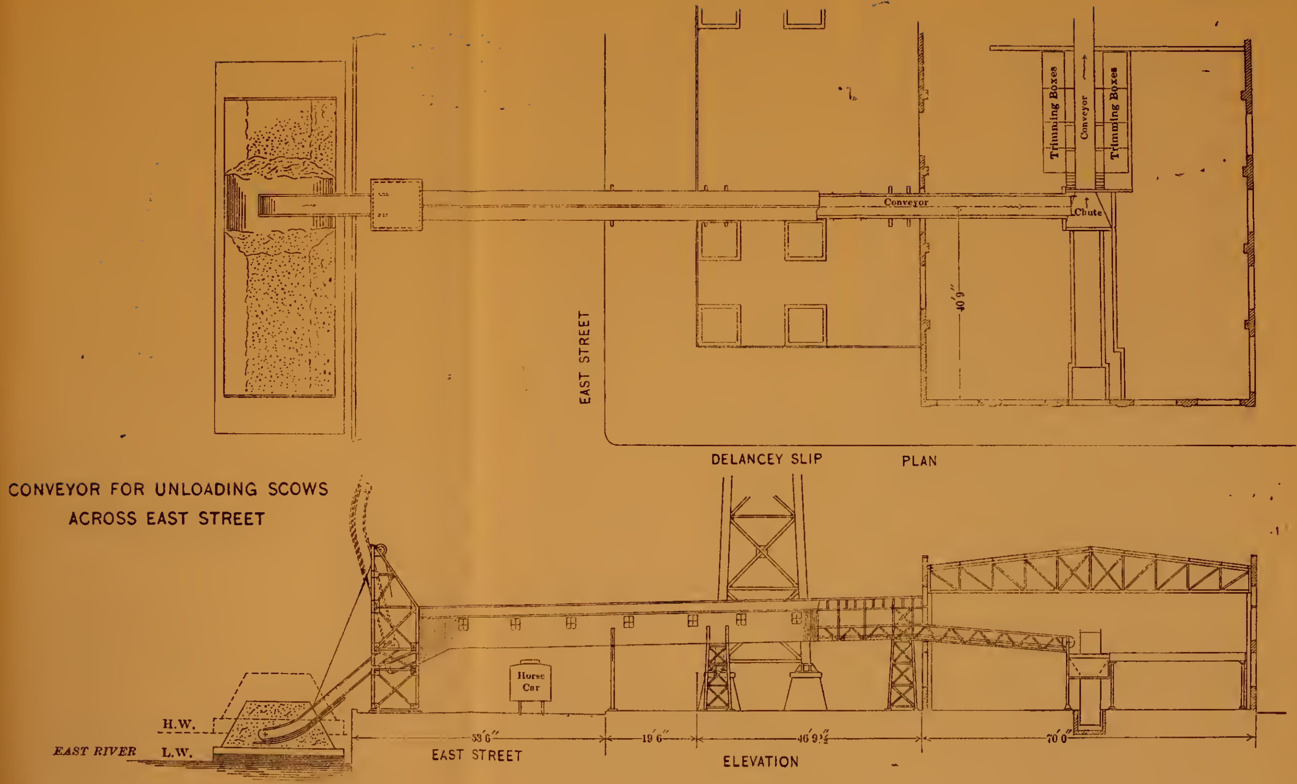
The privilege of picking the rubbish at the dumps in the Boroughs of Manhattan and the Bronx brought $71 000 for 1903. For a week in 1904 the amounts varied from $1 175 to $1 920. The collections in 1903 were 126 188 tons.

At the Atlantic Avenue incinerating plant in Boston, the amount picked out and sold averages about 25½% by weight. In New York, at the Thirtieth Street dump, the amount is about 43%; at the Forty-seventh Street dump, about 48%; and at the Delancey Slip station, about 32 per cent. The two latter places are equipped with conveyors which carry the material between the rows of pickers. At the other city dumps the amount picked out is not as large. As the Delancey Street station is equipped with an incinerating plant using the heat for steam generation, only about 30% is picked out, as the remainder is required for fuel, although with the conveyor and sorting opportunity it would be easy to pick out 50 per cent. The amount picked out varies with the market price for the paper, metals, etc.

IV.—Street-Sweepings.—The sweepings contain two constituents of value, namely, manure and paper. When these are mixed with the street dirt, they are valueless. If the manure could be collected separately, it might be sold for fertilizer, or be sold for the same purpose to the collectors of stable manures.

Considerable rubbish, often of the dirtiest sort, is found in public thoroughfares, where it has been thrown by a careless and shiftless population. These materials—rubbish, manure, paper, and dirt, the latter coming from the pavements and, therefore, varying with the kind in use—form the major part of the street-sweepings proper. There are some house sweepings collected in the streets, but this addition aggregates but a small part. In cities in which the streets

PLATE XXXIII.
PAPERS, AM. SOC. C. E.
APRIL, 1906.
PARSONS ON
REFUSE DISPOSAL.



are lined with trees, a considerable quantity of leaves is collected in the proper season, which increases the bulk materially.

TABLE 5.—Composition of Rubbish.

Percentages by Weight.

Component Parts.	Percentage Picked Out as Marketable.				Percentage of Total Composition.		
	City of New York.			Boston.	New York.	London.	Berlin.
	Delancey Slip station. (Parsons.)	Thirtieth St. dump. (Stearns.)	Forty-seventh St. dump. (Stearns.)	Atlantic Ave. station. (Morse.)	(Craven)*	(Russell)*	(Bohm and Grohn.) *
	%	%	%	%	%	%	%
Stoneware	...	...	...	...	...	5.0	33.5
Rags	4.6	...	2.78	0.76	15.5	3.6	6.3
Rubber	...	...	...	...	0.1	...	...
Leather	...	...	...	...	1.8	...	3.8
Straw	...	...	...	...	...	29.7	19.7
Wood	...	7.3	8.91	...	1.4	...	2.2
Metals	0.86	1.3	4.10	0.12	3.3	9.2	4.2
Glass	...	1.4	0.76	0.35	2.9	13.1	7.0
Bagging	...	...	0.39	...	...	...	...
Carpets	...	...	0.57	...	...	...	...
Shoes	...	...	0.39	...	...	...	...
Hats	...	...	0.03	...	...	...	...
Rope and String	...	...	0.23	0.12	...	...	...
Paper	25.4	33.3	...	23.90	75.0	39.4	23.3
Newspaper	...	...	10.94	...	...	...	...
Manila	...	...	2.64	...	...	...	...
Pasteboard	...	...	10.35	...	...	...	...
Mixed	...	...	6.16	...	...	...	...
Books	...	...	0.55	0.24	...	...	...
Total marketable	30.86	43.3	48.80	25.49	...	...	...
" worthless	69.14	56.7	51.20	74.51	...	...	...
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0

*International Engineering Congress, *Transactions*, Am. Soc. C. E., Vol. LIV., Part E, Paper by Rudolph Hering, M. Am. Soc. C. E.

The sweepings are combustible. The weight varies from 800 to 1 400 lb. per cu. yd., as much depends on the dryness of the weather at the time of collection.

Table 6 gives some idea of the composition of the street-sweepings.

The writer entered into correspondence with a number of cities in order to obtain data in regard to the annual collection of general refuse. It was found that many of the cities did not make separate collections of the different classes of refuse, and that many which did make separate collections did not keep records, or if they did

keep records, did not keep them with sufficient accuracy to be of service for comparison. For instance, Louisville, Ky., published in 1903 a very elaborate report in which the collections were given in cart loads. As no reference was made to the size of the carts, or their average loading, it was not possible to classify these collections.

TABLE 6.—COMPOSITION OF STREET-SWEEPINGS.

PERCENTAGES BY WEIGHT.

Component parts.	New York. (Craven.)	Washington. (Wiley.)	Berlin. (Vogel.)	London. (Letheby)†
Moisture	37	35	39	35
Organic matter	31	20	23	36
Ash	32	45	38	29‡
	100	100	100	100
Proportion of organic matter to ash	1 : 1	1 : 2.2	1 : 1.6	1 : 0.8

* From U. S. Dept. of Agriculture "Fertilizing Value of Street Sweepings," Bulletin No. 55, Division of Chemistry. International Engineering Congress, *Transactions*, Am. Soc. C. E., Vol. LIV, Part E, Paper by Rudolph Hering, M. Am. Soc. C. E.

† In dry weather.

‡ Powdered stone and abraded iron.

Some of the cities kept a record in yardage, and, in order to transform yardage into weight, the following schedule was used:

Weight of garbage	1 150 lb. per cu. yd.	
" " street-sweepings	850 " " " "	
" " ashes	1 350 " " " "	
" " rubbish	200 " " " "	

Table 7 gives statistics of the annual collection of general refuse, in tons of 2 000 lb.

Table 8 gives the monthly collections, in pounds per capita per day, in those cities where the records were available. This table shows some variation in regard to the amounts, which is especially noticeable in the column, "Average per day." This variation is caused, to some extent, by irregularity in classifying the collections —for instance, in some cities, rubbish and ashes are collected with the garbage—thus, in Philadelphia the record shows that the quantity of garbage per capita per day is nearly 1.2 lb., while in the other cities it varies from about 0.3 to 0.8 lb., while the ashes and rubbish collections are low when compared with the others.

TABLE 7.—Annual Collection of City Refuse, in Tons of 2 000 lb.

City.	Year.	Population.	Garbage.	Street-sweepings.	Ashes.	Rubbish.	Totals.
Borough of Manhattan	...	1 917 676	184 275	*B*	1 405 606	120 434	1 710 315
" " Bronx	...	268 341	13 475	...	109 699$\frac{1}{4}$	5 754	128 928
" " Brooklyn	...	1 291 597	75 675	...	501 888$\frac{1}{2}$	39 246	616 809
Total of three Boroughs of New York	1903.	*A* 3 477 614	273 425	...	*D* 2 017 193$\frac{3}{4}$	*C* 165 434	2 456 053
Buffalo, N. Y.	June 30, 1903.	*E* 391 148	*F* 23 803	*B*	*F* 150 469	*F* 15 750	190 022
Philadelphia, Pa. *H*	Dec. 31, 1903	*E* 1 385 549	301 643	93 044	425 650	13 975	834 312
Milwaukee, Wis. *H*	...	*E* 307 854	*K* 30 441	...	...	...	...
Cincinnati, Ohio *H*	1903.	381 200	21 600	39 362	142 507	...	...
Washington, D. C.	Dec. 31, 1903.	*E* 296 182	*M* 33 664	*L* 31 810	*L* 83 078	*L* 14 150	...
Newark, N. J. *H*	...	*E* 272 149	15 152	33 496	*G* 208 050	*G*	...
Cleveland, Ohio *J*	Dec. 1, 1904.	425 000	43 680	35 036	...	...	...
Pittsburg, Pa.	...	320 000	*O* 47 000	...	...	...	...
St. Louis, Mo.	Dec. 31, 1903.	*E* 622 350	62 400	...	*N*	*N*	...

A. Board of Health census, 1903.
B. Street-sweepings included with "ashes."
C. The Department of Street Cleaning estimates the weight of a cubic yard of rubbish and paper at 237 lb. F. L. Stearns, Assoc. M. Am. Soc. C. E., measured and weighed 96 loads in which 1 cu. yd. weighed 140 lb. This figure has been used in reducing the yardage to pounds, one load = 1 000 lb.
D. Including "steam" ashes from manufacturing plants.
E. United States census.
F. Report, Department of Public Works, Buffalo, 1903.
G. Rubbish and paper included with ashes.
H. Bureau of Street Cleaning, or Board of Public Works, 1903.
J. Bureau of Street Cleaning, or Board of Public Works, 1904.
K. Average for 1901 and 1903.
L. Average for 1903–1904.
M. Garbage for 1903; street-sweepings, ashes and rubbish averaged for 1903–1904.
N. Collections not made by the city.
O. Contractor's statement for 1903.

TABLE 8.—(*Continued.*)

City.	Classification.	Jan.	Feb.	March.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Average per Day.
Cincinnati, Ohio. *A*	Garbage													0.310
	Street-Sweepings													0.566
	Ashes													2.048
	Rubbish													
	Totals													2.924
Washington, D. C. *A*	Garbage	0.511	0.512	0.586	0.641	0.600	0.577	0.666	0.871	0.819	0.733	0.552	0.543	0.635
	Street-Sweepings													0.597
	Ashes													1.557
	Rubbish													0.264
	Totals													3.053
Newark, N. J. *B*	Garbage													0.305
	Street-Sweepings													0.674
	Ashes													4.189
	Rubbish													
	Totals													5.168
Cleveland, Ohio. *B*	Garbage													0.563
	Street-Sweepings													0.451
	Ashes													
	Rubbish													
	Totals													
Pittsburg, Pa.	Garbage													0.805
	Street-Sweepings													
	Ashes													
	Rubbish													
	Totals													
St. Louis, Mo.	Garbage													0.549
	Street-Sweepings													
	Ashes													
	Rubbish													
	Totals													

A—Report, Department of Street Cleaning. *B*—Report, Department of Public Works.

TABLE 8.—COLLECTION OF CITY REFUSE, IN POUNDS PER CAPITA PER DAY.

City.		Classification.	Jan.	Feb.	March.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Average per Day.
New York City.	Bor. Manhattan. *A*	Garbage	0.354	0.372	0.477	0.540	0.567	0.613	0.596	0.622	0.628	0.613	0.505	0.416	0.525
		Street-Sweepings	......	......	......	......	......	......	......	......	......	......	......	......	......
		Ashes	5.282	5.089	4.778	4.123	3.588	3.528	3.326	2.975	3.167	3.437	4.017	5.082	4.016
		Rubbish	0.294	0.277	0.311	0.345	0.389	0.391	0.385	0.351	0.365	0.377	0.337	0.317	0.345
		Totals	......	......	......	......	......	......	......	......	......	......	......	......	4.886
	Bor. Bronx. *A*	Garbage	0.185	0.194	0.249	0.282	0.296	0.319	0.311	0.324	0.328	0.320	0.263	0.217	0.274
		Street-Sweepings	......	......	......	......	......	......	......	......	......	......	......	......	......
		Ashes	2.932	2.833	2.653	2.294	1.994	1.962	1.843	1.651	1.762	1.912	2.235	2.822	2.241
		Rubbish	0.100	0.094	0.106	0.118	0.133	0.133	0.131	0.120	0.124	0.128	0.115	0.108	0.1175
		Totals	......	......	......	......	......	......	......	......	......	......	......	......	2.6325
	Bor. Brooklyn. *A*	Garbage	0.216	0.236	0.280	0.330	0.344	0.374	0.364	0.380	0.383	0.374	0.308	0.265	0.321
		Street Sweepings	......	......	......	......	......	......	......	......	......	......	......	......	......
		Ashes	2.783	2.684	2.520	2.176	1.892	1.863	1.754	1.565	1.673	1.813	2.121	2.680	2.127
		Rubbish	0.141	0.134	0.150	0.167	0.188	0.189	0.186	0.170	0.176	0.182	0.162	0.153	0.1665
		Totals	......	......	......	......	......	......	......	......	......	......	......	......	2.6145
Buffalo, N. Y. *B*		Garbage	0.244	0.230	0.268	0.279	0.299	0.333	0.389	0.405	0.486	0.445	0.340	0.290	0.334
		Street-Sweepings	......	......	......	......	......	......	......	......	......	......	......	......	......
		Ashes	2.762	2.662	2.501	2.161	1.874	1.850	1.741	1.552	1.660	1.800	2.101	2.660	2.110
		Rubbish	0.188	0.178	0.200	0.222	0.250	0.252	0.247	0.226	0.227	0.242	0.217	0.203	0.221
		Totals	......	......	......	......	......	......	......	......	......	......	......	......	2.665
Philadelphia, Pa. *A*		Garbage	0.878	0.849	0.753	1.011	0.812	0.976	1.327	1.340	2.494	1.479	1.167	1.216	1 192
		Street-Sweepings	0.281	0.249	0.350	0.437	0.404	0.355	0.463	0.349	0.484	0.373	0.370	0.310	0.368
		Ashes	2.075	2.106	1.817	2.136	1.513	1.351	1.465	1.101	1.459	1.295	1.608	2.278	1.683
		Rubbish	0.071	0.061	0.047	0.061	0.052	0.050	0.058	0.046	0.062	0.018	0.049	0.055	0.055
		Totals	......	......	......	......	......	......	......	......	......	......	......	......	3.298
Milwaukee, Wis. *B*		Garbage	0.539	0.514	0.511	0.491	0.514	0.594	0.636	0.625	0.588	0.576	0.489	0.422	0.542

TABLE 9.—AVERAGE COLLECTIONS PER CAPITA PER DAY.

	Pounds.	Yards.	Percentage.	
			By weight.	By volume.
Garbage	0.5296	0.000460	14.12	11.77
Street-Sweepings	0.5310	0.000625	14.15	15.99
Ashes	2.4960	0.001849	66.54	47.31
Rubbish	0.1948	0.000974	5.19	24.93
Total	3.7514	0.003908	100.00	100.00

Table 9 was made by taking the averages of the figures in Table 8. Some of the cities collected two of the divisions together and gave the result under one division; thus, New York collected the street-sweepings and ashes together and gave the total as ashes.

Table 10 was made up by using the foregoing percentages to divide those double collections, and gives the averages for all the cities. It is, therefore, more nearly correct than Table 9.

TABLE 10.

	Pounds	Yards.	Percentage.	
			By weight.	By volume.
Garbage	0.5296	0.000460	15.27	12.13
Street-Sweepings	0.4992	0.000587	14.39	15.48
Ashes	2.2290	0.001696	64.28	44.71
Rubbish	0.2101	0.001050	6.06	27.68
Total	3.4679	0.003793	100.00	100.00

The average collections per capita per day for the cities mentioned are given in Table 9.

The figures given in Table 8, for the monthly collections of general refuse, in pounds per capita per day, are shown graphically in Fig. 1.

In the same manner, Figs. 2 to 4 show graphically the collections of ashes, garbage and rubbish. All these diagrams show a characteristic maximum about January and a minimum about

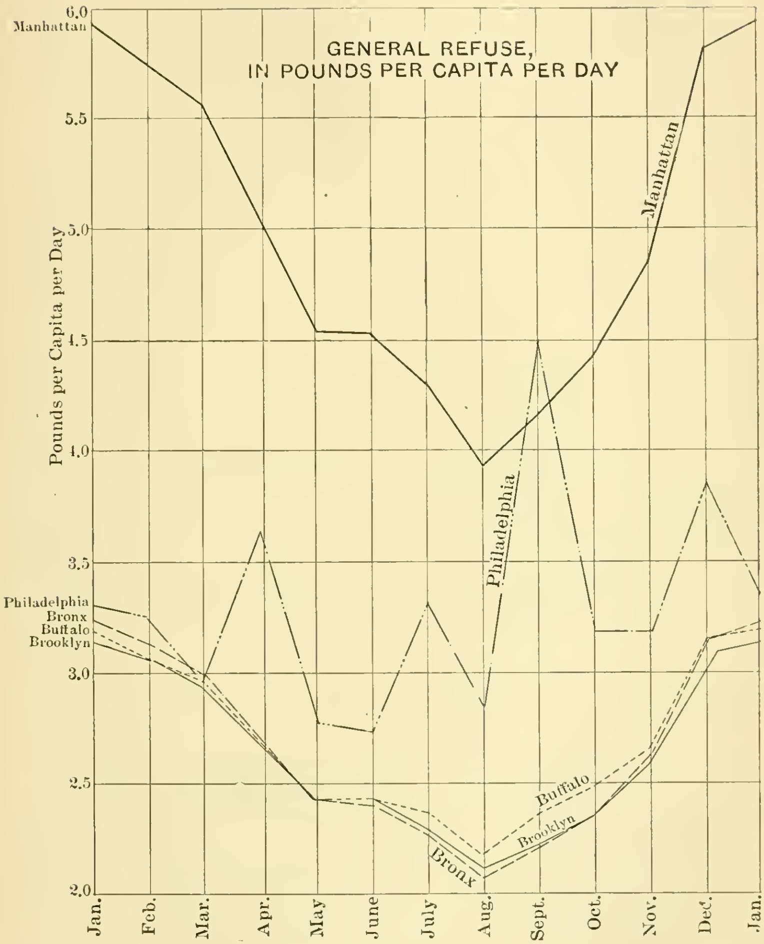

GENERAL REFUSE,
IN POUNDS PER CAPITA PER DAY
Pounds per Capita per Day
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
Manhattan
Philadelphia
Bronx
Buffalo
Brooklyn
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Jan.

FIG. 1.

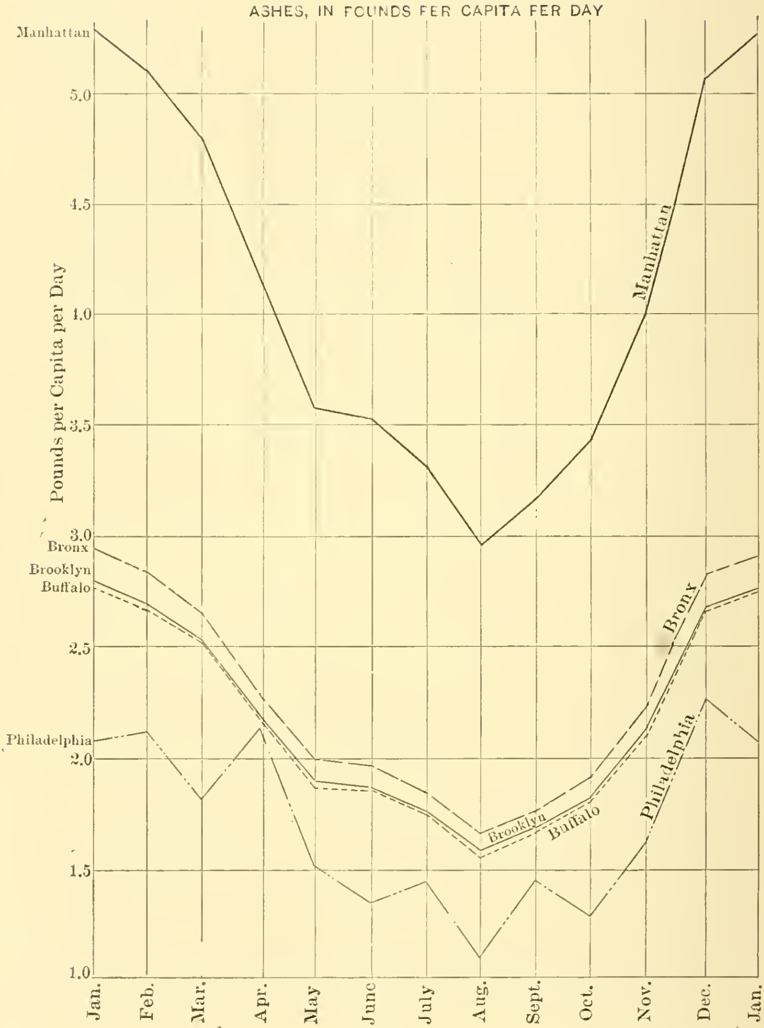

ASHES, IN POUNDS PER CAPITA PER DAY
Pounds per Capita per Day
Manhattan
5.0
4.5
4.0
3.5
3.0
Bronx
Brooklyn
Buffalo
2.5
Philadelphia
2.0
1.5
1.0
Manhattan
Bronx
Brooklyn
Buffalo
Philadelphia
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Jan.

FIG. 2.

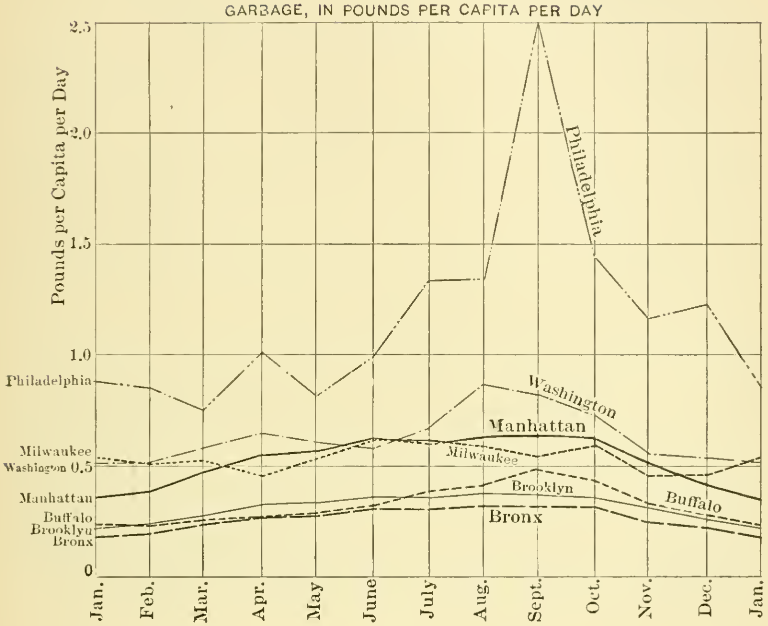

GARBAGE, IN POUNDS PER CAPITA PER DAY
Pounds per Capita per Day
2.5
2.0
1.5
1.0
0.5
0
Philadelphia
Milwaukee
Washington
Manhattan
Buffalo
Brooklyn
Bronx
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Jan.

Fig. 3.

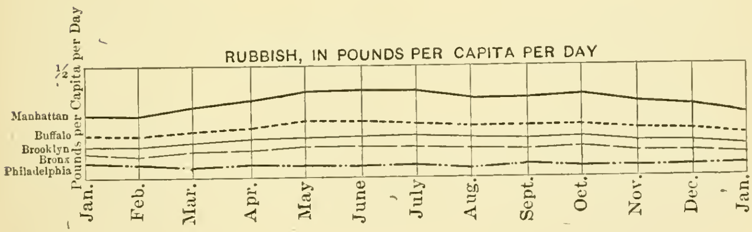

RUBBISH, IN POUNDS PER CAPITA PER DAY
Pounds per Capita per Day
½
Manhattan
Buffalo
Brooklyn
Bronx
Philadelphia
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Jan.

Fig. 4.

August. The maxima and minima are evidently caused by the collection of ashes, because the rubbish collections, while much more uniform than garbage or ashes, are in maximum during the summer months, and the garbage is in maximum about September.

Fig. 5 shows the monthly collections, in pounds per capita per day, for the Borough of Manhattan, New York City.

In studying these diagrams, it must be borne in mind that the ash collections as reported for the Borough of Manhattan, New York City, also contain the street-sweepings. The curve, therefore, representing Manhattan in Fig. 2 is too high by just the amount of the street-sweepings, which are not reported separately. In the same manner, in Fig. 3, the curve representing garbage collections for Philadelphia is too high, as some rubbish and ashes are collected and reported under the head of garbage.

In Fig. 1, the curve representing Manhattan (as well as the figures in Table 8 for the Borough of Manhattan) is higher than those of the other cities reported, which can be accounted for by the fact that the territory of the Borough of Manhattan is completely built over, and the whole population is served by the collection carts. In the other cities there are outlying districts which are included in the population, but which are not served by the collection carts. It is probable, therefore, that the actual figures for Manhattan are more accurate than for some of the other places.

The Fuel Value of Refuse.

The value of refuse as a fuel depends on the combustible matter which it contains. There does not appear to be much difference in the general make-up of the combined refuse as collected in America and Europe. It differs more in quantity than in complection.

Garbage will burn, as can be proved by throwing it into a range or stove. It requires enough heat to evaporate the water, and then the dry matter will burn of itself and assist in drying the next charge. This dried portion will generate about 7 500 B. t. u. per lb.

The combustible in the ashes has a total heat of combustion varying from, say, 10 000 to 14 000 B. t. u. per lb. Therefore, the dry ash, on a basis of 25% of combustible, will generate about 3 000 B. t. u. per lb. As the ashes, according to the analyses given


GENERAL REFUSE, IN POUNDS PER CAPITA PER DAY
BOROUGH OF MANHATTAN, CITY OF NEW YORK. YEAR 1903.
ASHES, AND STREET-SWEEPINGS
GARBAGE
RUBBISH
Pounds per Capita per Day
Pounds per Capita per Day
5.0
4.0
3.0
0.6
0.5
0.4
0.4
0.3
Jan.
Mar.
May
July
Sept.
Nov.
Jan.

FIG. 5.

before, contain an average of more than 25% of combustible, there is an allowance at this latter figure for some moisture.

The rubbish, being composed principally of paper, rags and wood, has a total heat of combustion varying from, say, 5 000 to 7 500 B. t. u. per lb.

The street-sweepings contain some combustible material, which can be taken as having a total heat of combustion of about 6 000 B. t. u. per lb. The remainder of the street-sweepings consists of incombustible material and water. If it be assumed that 33% is incombustible, 42% combustible, and 25% water, the burning of the street-sweepings would generate about 2 000 B. t. u. per lb., a probably safe figure.

Assume that the combined collection is composed as shown in Table 11:

TABLE 11.

Classification.	City collections.	Used as fuel.
Garbage	15.0% (70% moisture)	22.9%
Ashes	64.0% (25% combustible)	48.7%
Rubbish	7.0% (33⅓% sorted out)	7.1%
Street-sweepings	14.0%	21.3%
Totals	100.0%	100.0%

Also, assume that 33⅓% of the rubbish is sorted out and only half the ashes are delivered to the furnace, so that the percentage of the constituents as thrown into the furnace would be in the ratio stated in the last column of Table 11. From these assumptions, a heat balance can be worked out as shown in Table 12.

The combustible portion is: garbage, 0.0687; ashes, 0.0913; rubbish, 0.064 (allowing 10% for the incombustible portion); and street-sweepings, 0.071 (allowing 33% as incombustible); making a total of 0.295 lb.

If the heat is passed through a boiler, and the losses be assumed at 40%, then the available useful heat would be 2 391 $\times$ 0.60 = 1 434 B. t. u., or sufficient to evaporate 1.48 lb. of water from and at 212° fahr.

If 1 lb. of coal can evaporate 9 lb. of water, and is worth $3 per

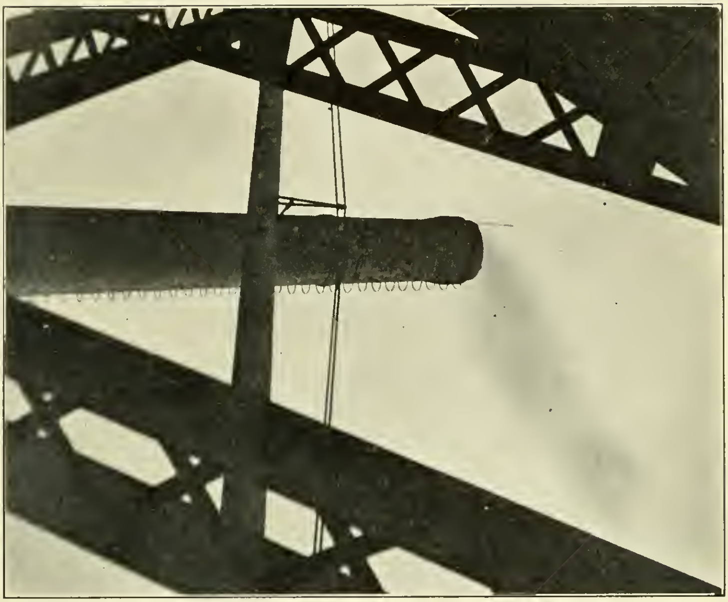


FIG. 1.—SMOKE FROM RUBBISH INCINERATOR, WHEN BURNING DRY RUBBISH.

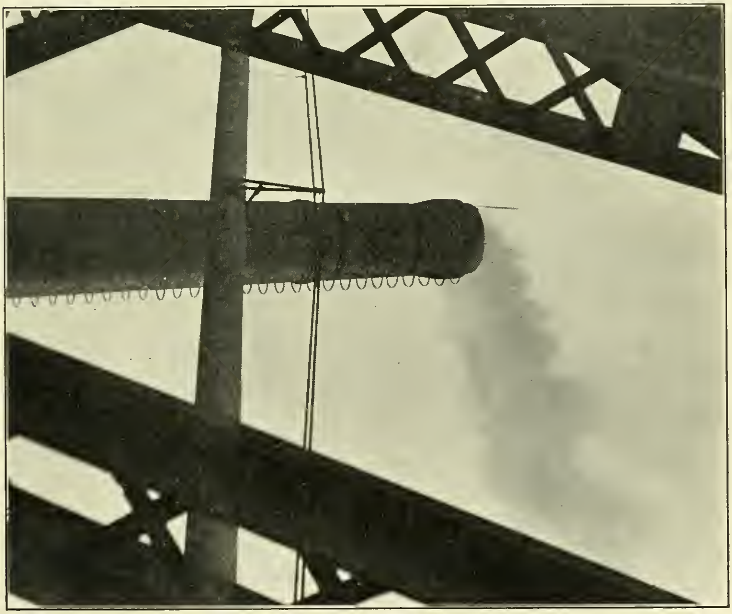


FIG. 2.—SMOKE FROM RUBBISH INCINERATOR, WHEN BURNING WET RUBBISH.

ton, then the value of this refuse as a fuel would be (\$1.48 ÷ 9) × 3.00 = \$0.49 per ton.

TABLE 12.—Estimate for Heat Balance.

Temperature of Air, 60° fahr. Temperature of Furnace, 2 000° fahr. Fuel: City Refuse, Combined Collections—One Pound, moderately dry.

		Total heat of combustion. B. t. u.	Dissipation of heat. B. t. u.
Garbage:	Dried portion: 0.229 × 0.30 × 7 500	515	
	Moisture; 0.229 × 0.70 × [(212—60) + 966 + 0.48 (2 000 — 212)]		317
Ashes:	On the basis that only three-fourths of the combustible is burned; then 0.487 × 0.75 × 3 000	1 095	
Rubbish:	On the basis that one-third has been sorted out; then 0.071 × 5 000	355	
Street-sweepings:	On the basis that there is 33% incombustible matter; then 0.213 × 2 000	426	
Heating fuel mass:	Taking the specific heat of the mass at 0.2; then 1 × 0.2 × (2 000 — 60) = 388, of which, say, one-third is lost through raking out the hot ashes		129
Heating air supply:*	On the basis of 4.2 lb. of air per lb. of fuel; then, 4.2 × 0.2375 (2 000 — 60). This is about 14.3 lb. of air per lb. of combustible.		1 945
	Estimated British thermal units per pound	2 391	2 391

* Pounds of air per pound of fuel calculated by difference.

It would be possible to vary the proportions of the constituents fed to the furnace, as long as sufficient combustible material was supplied to evaporate the moisture in the garbage. This condition can be attained by practice at the furnace. It is also self-evident that an increased efficiency will be obtained if the air supplied be heated.

In the same manner, another heat balance can be worked out for the burning of garbage, rubbish and street-sweepings, when all the ashes are kept out of the furnace and utilized for land filling.

Taking the same general analysis as before, for the combined collections, the fuel proportions would be as shown in Table 13.

Assume as before that one-third of the rubbish is sorted out as marketable; then the percentage of the constituents as thrown into the furnace would be in the ratio stated in the last column of Table 13.

TABLE 13.

Classification.	City collections.	Used as fuel.
Garbage	15% (70% moisture)	44.5%
Rubbish	7% (33⅓% sorted out)	13.9%
Street-sweepings	14%	41.6%
		100.0%

TABLE 14.—Estimate for Heat Balance.

Temperature of Air, 60° fahr. Temperature of Furnace, 2 000° fahr. Fuel: City Refuse—Garbage, Rubbish and Street-Sweepings, One Pound.

	Total heat of combustion. B. t. u.	Dissipation of heat. B. t. u.
Garbage: Dried portion; 0.445 × 0.30 × 7 500	1 001	
Moisture; 0.445 × 0.70 × [(212 — 60) + 966 + 0.48 (2 000 — 212)]		615
Rubbish: 0.139 × 5 000	695	
Street sweepings: 0.416 × 2 000	832	
Heating fuel mass: 1 × 0.2 × (2 000 — 60), of which about one-third is lost		129
Heating air supply: On the basis of 3.9 lb. of air per lb. of fuel; 3.9 × 0.2375 × (2 000 — 60). This is about 9.9 lb. of air per lb. of combustible		1 784
Estimated British thermal units per pound	2 528	2 528

The combustible portion is: garbage, 0.1335; rubbish, 0.1251 (allowing 10% for the incombustible portion); and street-sweepings, 0.1386 (allowing 33% as incombustible); making a total of 0.3972 lb.

If the heat is passed through a boiler, and the losses are assumed at 40%, then the available useful heat would be 2 528 × 0.60 = 1 516 B. t. u., or sufficient to evaporate 1.57 lb. of water from and at 212° fahr.

On the basis of 34.5 lb. of water evaporated from and at 212° fahr., as equivalent to 1 b. h. p., these two results would represent a boiler horse-power for about 23.3 lb. of combined collections and 22.0 lb. of mixed garbage, rubbish and street-sweepings.

FIG. 1.—FRONT OF EAST FURNACE, SHOWING STOKING DOORS.

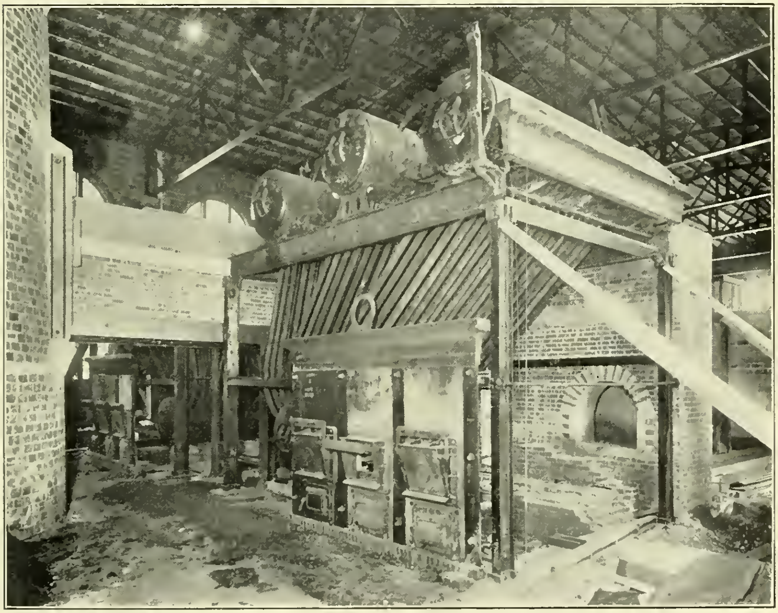


FIG. 2.—BOILERS FOR WILLIAMSBURG BRIDGE LIGHTING STATION AND RUBBISH INCINERATOR PLANT.

The results also indicate that, in order to obtain a high furnace temperature, an artificial draft is needed, so as to get the benefit of a thorough mixture of the oxygen with the fuel and not require too great a surplus of air. In practice, the highest evaporative results per pound of collection burned have been obtained when the boiler is set directly over the fires, as the heat is direct and the losses are reduced to a minimum. Such a plan, however, is not conducive to the maintenance of high temperatures in the furnace, and, consequently, the complete destruction of the mass by thorough combustion is apt to be sacrificed. As the latter consideration is of paramount importance, the boilers should be set to receive the gases of combustion after the combustion has been completed, and while still at high temperature.

The experience obtained in England by the burning of mixed collections shows an evaporation varying from ½ to 2 lb. of water from and at 212° fahr. per lb. of refuse. The best results are obtained by the use of artificial drafts, so as to obtain the benefit of a thorough mixture of the gases with the smallest air supply. The air supplied in the blasts is about 3½ lb.* per lb. of average mixed refuse.

Table 15 gives some figures relative to the total heats of combustion of the constituent elements of refuse.

TABLE 15.—Approximate Calorific Values of Refuse.

	British thermal units, per pound.	Authority.
Garbage, dry	7 500	H. de B. Parsons.‡
" as collected	800	" ‡
Bones and offal, dry	8 000	Dawson.†
" " " average moisture	5 333	"
Ashes, combustible portion	12 000	H. de B. Parsons.
" as collected, average	3 000	"
Rubbish. from	7 500	" ‡
to	5 000	" ‡
Paper, straw, fibrous matter, and vegetable refuse, dry	3 800 §	Dawson.†
Paper, straw, fibrous matter, and vegetable refuse, average moisture	2 500 §	"
Rags, dry	5 000	"
" average moisture	3 333	"
Wood, dry	7 800	H. de B. Parsons.
" average moisture	6 500	"

* George Watson. *Transactions*, Am. Soc. M. E., Vol. XXV, 1904.

† "Disposal of Municipal Refuse," *Transactions*, Am. Soc. C. E., Vol. LIV, Part 1905.

‡ Estimated from its composition, and verified by its behavior in the incinerators.

§ The writer considers this too small.

The Rubbish Incinerator.

In New York the city refuse is separated into classes and collected by different carts. The rubbish collections include paper of all kinds, books, magazines, cardboard, rags, barrels, boxes, crates, shoes, hats, pieces of leather, rubber, cans, metals, garments, beds, bedding, mattresses, bed springs, sofas, chairs, broken furniture of all kinds, and various other articles, both large and small, of almost every conceivable nature. The material is heterogeneous and bulky. As packed in the collection carts, its weight averages 141 lb. per cu. yd., which is the average of the actual weighing of 135 carts from different districts.

As this material makes a very poor land-fill, and as it cannot be dumped at sea without danger of its floating back on the beaches, the rubbish incinerator plant was designed by the writer for the Department of Street Cleaning, in order to reduce its bulk, to transform it into ash which can be used for land-fill, to provide better picking facilities than is afforded by dumping directly into scows, Fig. 1, Plate XXX, and to destroy in a sanitary manner the beds, bedding, mattresses and furniture, some of which may be carriers of disease.

Location.—The plant is located on city property, beneath the Williamsburg Bridge, facing Delancey Slip, between East Street and Tompkins Street, Borough of Manhattan. It is thus close to the East River, being separated therefrom only by the width of East Street.

General Arrangement.—The arrangement of the plant is shown on Plate XXXI. The collection carts drive into the building through any of the four doors on the Delancey Slip front, and dump their loads on the belt conveyor. If the material arrives faster than it is wanted on the conveyor, the loads are dumped on the floor of the receiving room, the material then being pushed upon the conveyor by hand labor as desired. The empty carts go back for more material.

The conveyor carries the material between two rows of "trimmers," as the men are called who pick out the marketable material and place the different kinds in separate boxes. This marketable material is baled by presses and by hand, and removed by the contractor who pays the city for the privilege.

The material not picked passes to the end of the conveyor, where it is dumped on a steel charging floor, and pushed into the furnaces by furnace men.

The street-sweeping carts drive up an inclined runway and dump directly on the charging floor, thus preventing the street dirt from injuring the salable portion of the rubbish.

The furnaces are connected with a stack, and, by means of dampers, the hot gases may be diverted either directly to the stack or through the boilers.

On one side of the building doors are provided for carts to enter to remove the ashes and the baled material without interfering with the material being received.

A second floor over part of the building provides storage space for the material to be burned on holidays when there is no collection, and for night use. The receiving floor also provides additional space for storage, as well as the yards on both sides of the building, and a scow, filled at some other dumping station, is, furthermore, kept at the wharf opposite the building. In this way there is no trouble from lack of material to burn.

Building.—The building is 150 ft. deep and 70 ft. wide. The ground floor is at the level of the sidewalk, and is paved with Belgian blocks. Over the northern half and along the west side there is a second floor. This upper floor, used for storage, is reached by an inclined driveway outside the structure.

The front of the building, Fig. 2, Plate XXX, is of brick, with buff brick facing, and the sides are of Phœnix, hollow, tile-block construction, stuccoed on the outside. The pilasters beneath the roof trusses are of red brick, locked into the tile blocks, course and course.

The roof is supported by steel trusses and purlins, and is covered with saturated roofing felt, coal-tar pitch and gravel. The upper floor is of reinforced concrete on steel beams and girders, supported on steel pipe columns.

The windows are as large as could conveniently be made, consistent with strength in the walls.

A division wall of tile blocks, carried to the upper floor, divides the receiving space from the furnace space, and keeps the combustible material away from the fires. This wall does not reach en-

tirely across the building, and the space is used as a driveway for ash carts, which can enter at one of the side doors and pass out at the other.

Conveyor.—The conveyor is of the metallic, apron type, 48 in. wide, with side angles to hold the rubbish. The flights are each 6 in. long and the full width of the conveyor, and are lapped over each other, so that there is no space between them as they travel over the head shafts. Fig. 1, Plate XXXII, shows the conveyor as it rises from the receiving floor to the tops of the furnaces, and passes between the sorting boxes.

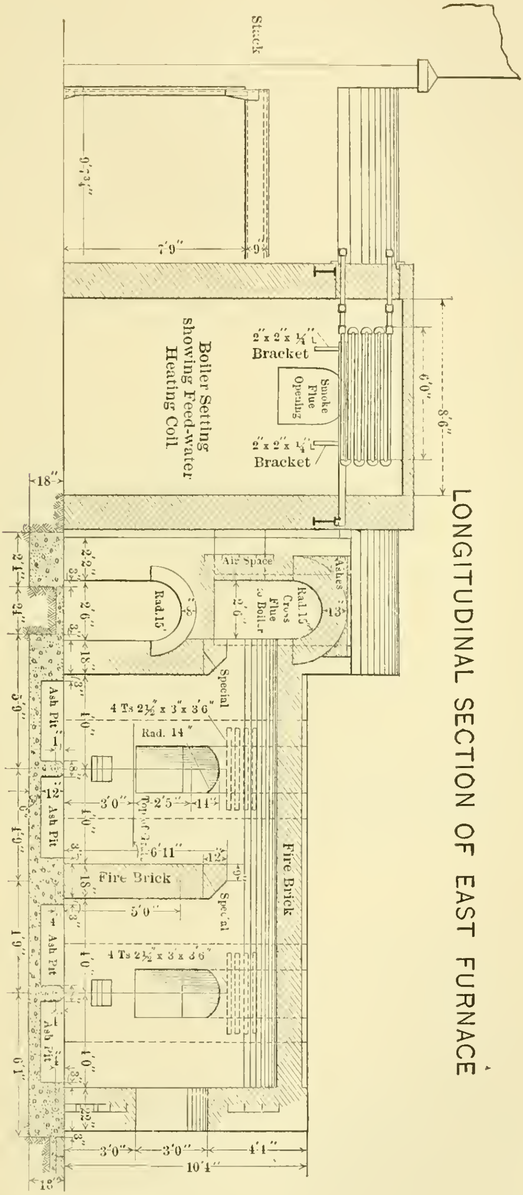
By spreading out the material on the conveyor the opportunity for picking is greatly facilitated. The picking and sorting privilege is let to a contractor, who pays a sum sufficient to cover more than all the labor charges in the plant. The amount of material picked out varies with the market value for the old paper, cans, etc., between 25 and 40% by weight of the material delivered. Some of this material, baled and ready for removal, is shown in Fig. 3, Plate XXX.

To facilitate the unloading of scows, which bring rubbish from other city districts, a conveyor across East Street has been designed, but not yet constructed, Plate XXXIII. This will unload the material on the upper floor, from which it will be pushed through a chute to the main conveyor.

Both these conveyors are arranged for power drives, using steam from the boilers. The speed of the conveyor is controllable, but the working speed is about 50 ft. per min.

Charging Platform.—The platform over the furnaces is of steel construction, covered with steel floor-plates. The conveyor dumps upon this platform, Fig. 2, Plate XXXII.

Furnaces.—There are two furnaces, of somewhat different design. The west furnace was designed by F. L. Stearns, Assoc. M. Am. Soc. C. E., and has two grates for part of its length. The area of the upper grate is 113 sq. ft. The east furnace was designed by the writer, and has one grate, 74 sq. ft. in area, divided into two cells by a division wall, as shown in Fig. 6. The material is fed through two chutes at the back, is stoked forward on the grate as wanted, and the clinkers are pulled out through the stoking doors, Fig. 7. The motion of the material is thus continuous. This sys-



LONGITUDINAL SECTION OF EAST FURNACE

FIG. 6.

tem has worked well, with the exception of the damage to the fire-brick lining due to the stoking tools, and improvements are planned to remedy this trouble.

Both furnaces are of brick, with fire-brick lining, and are strongly tied with stays made of 8-in. channels set in pairs.

The hot gases pass through a cross-flue, with dampers, either to the boilers or to the stack. The dampers are of special tile, and necessarily very large, Fig. 8. They are raised and lowered by triplex blocks and chains.

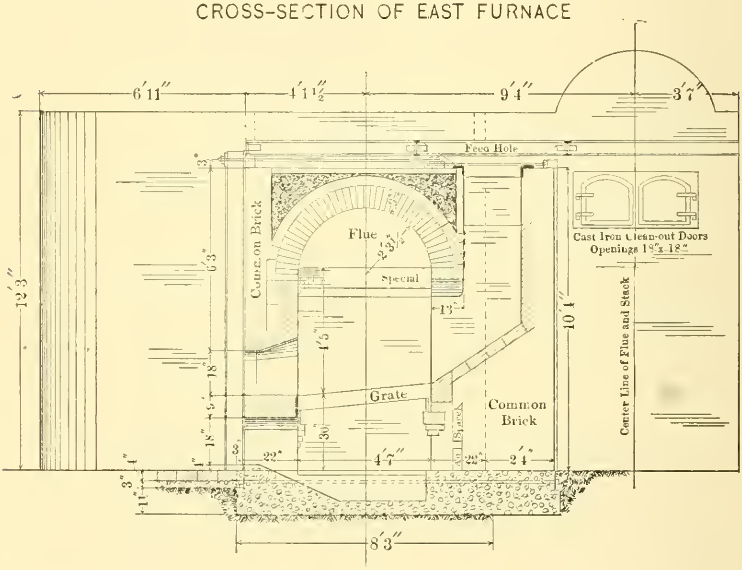
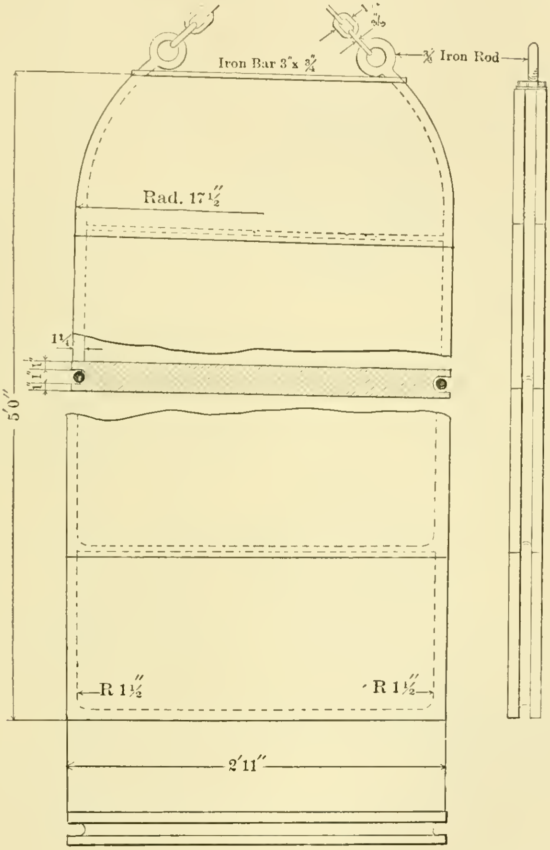


Fig. 7.

The material burns freely, often consuming at the rate of 40 lb. per sq. ft. of grate per hour. Even when wet, that is, rainy day collections, or when wet street-sweepings are fed in, the effect is not detrimental to rapid combustion. When dry, the smoke from the stack has a light bluish color, and when wet a light yellowish tinge. Rarely does the smoke exceed the amount shown in Figs. 1 and 2, Plate XXXIV. These photographs were taken when the furnaces were burning about 35 lb. per sq. ft. of grate per hour, the former when the rubbish was dry and the latter when it was wet.

On the end of each furnace there is a large door, to feed in beds, furniture and similar bulky articles. Fig. 1, Plate XXXV, shows the stoking doors on the front of the east furnace.

Boilers.—The boilers are Stirling water tubular, Fig. 2, Plate



TILE DAMPER USED IN FLUE

FIG. 8.

XXXV, and each contains 1 890 sq. ft. of effective heating surface. The tubes are 3¼ in. in diameter. Each boiler has its regular coal grate, and as the hot gases from the rubbish furnaces enter above these grates, they are always ready for coal firing (by simply arrang-

ing the dampers), should any accident happen to the fuel supply. The coal firing doors are placed on one side of the boiler setting, instead of at the end, as usual, a change made to accommodate the arrangement of the plant.

Each boiler has a feed-water economizer coil of brass pipe, set in the flue to the stack. These coils are connected with unions and can be withdrawn at any time without stopping the plant.

Stack.—The stack is of hollow, radial-block construction, built by M. W. Kellogg and Company. It is 200 ft. high, so that the top would be 75 ft. above the roadway of the bridge. The outside diameter at the base is 17 ft. and at the top 5 ft. 9 in. The stack is lined to a height of 135 ft. above the foundation. The lining above the smoke inlet is of unusual construction, being in the form of a continuous helical band, 16 ft. 5 in. high at any point. It is supported on corbels, but, being arranged helically, there are only three corbel blocks in each horizontal row of the outer column. The object of this helical arrangement is to prevent the outer column from cracking, when exposed to sudden changes in temperature, due to continuous corbeling in horizontal courses. This arrangement is shown in Fig. 9.

The foundation is of concrete, supported on piles.

The Electric Lighting Station.

Building.—In order to utilize the steam generated, an electric lighting station was built by the Department of Bridges, from plans by the writer. The building is 60 ft. deep and 50 ft. wide, and is separated from the incinerator building by 20 ft.

It is of steel skeleton construction, with the trusses supporting a reinforced cinder-concrete roof, covered with felt and coal-tar pitch and gravel. The curtain walls are of brick, with large windows, and are made to match the other building.

The north half of the building has a second floor of reinforced cinder-concrete construction, designed to be used at some future time for a storage battery plant. The general arrangement is shown in Plates XXXI and XXXVI.

Machinery.—There are two 100-k.w. and one 50-k.w. direct-connected units. The generators are multipolar, direct-current ma-

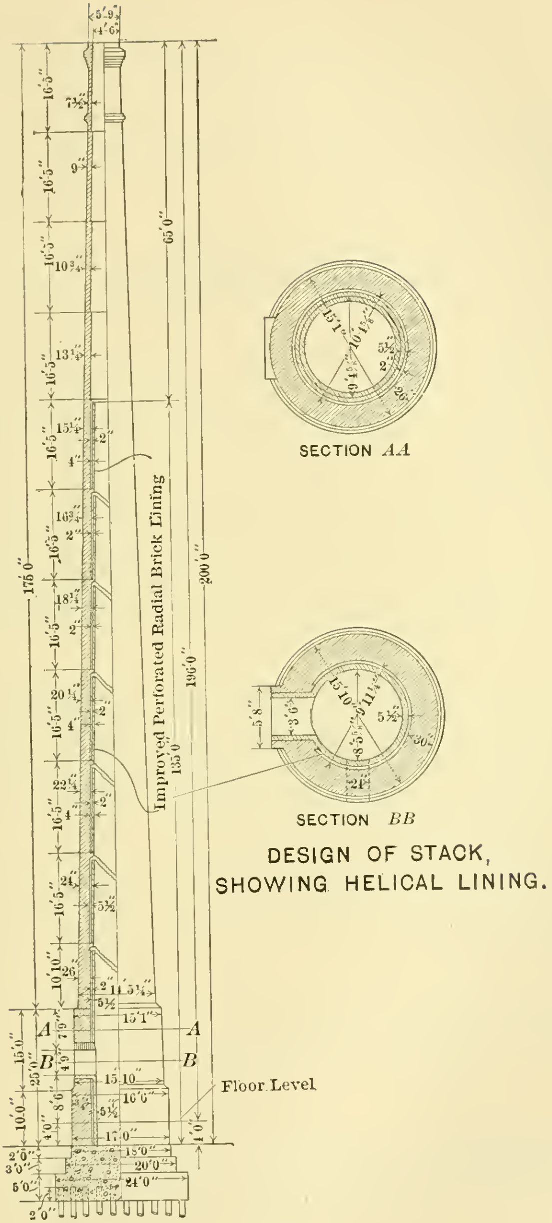

Improved Perforated Radial Brick Lining
SECTION AA
SECTION BB
DESIGN OF STACK,
SHOWING HELICAL LINING.
Floor Level

Fig. 9.

chines, built by the Burke Electric Company, and arranged for the three-wire system. They are all wound for 250 volts, with 125 volts on either side.

The generators are driven by "Ideal," cross-compound, horizontal, condensing engines, the large units having cylinders 12 and 20 in. in diameter with 12-in. stroke, and the small unit 9 and 16 in. in diameter with 10-in. stroke.

Each unit has an independent Blake, vertical beam, jet condenser, for which the suction and discharge are connected to the East River. The exhausts are all by-passed, and extend through the roof, so that the engines can be operated as non-condensing.

The exhausts from the condensers and feed pumps are passed into an open feed-water heater of the "Cockrane" type, into which are also returned the discharges from all traps that do not handle oil or grease.

Piping.—The piping is as simple as possible. The steam main is 7 in. in diameter, graded down from the boilers, with the far end turned down to form a pocket for drainage of condensation, and trapped. The supply branches are all taken from the top, and expansion is provided for by easy bends.

Current.—The electricity generated is used to light the Williamsburg Bridge, the incinerator building and the lighting station.

The cables are in underground conduits to the foot of the intermediate tower of the bridge, whence they are carried up the tower to the electric distribution of the bridge.

The number of lights, etc., at present connected, is given in Table 16, but when the storage plant is completed, the electric station can have a day load, and the current thus stored can be used for additional work as well as to assist the bridge load in cases of accident.

TABLE 16.—LIGHTS, ETC., SUPPLIED BY LIGHTING STATION.

Lights, etc.	Bridge.	Incinerator.	Station.
2 000-c. p. arc lamps	168	8	4
16-c. p. incandescent lamps	707	40	20
Electric heaters	20	....	....
Electric motors	3	....	....

The plant was formally opened by Mayor McClellan on October 30th, 1905. The cost of the plant was as follows:

Incinerator, including building, stack, east furnace, and equipment	$26 768
West furnace	4 000
Runway	1 550
Conveyor	1 875
Lighting station, including building, all machinery, boilers, and electrical equipment.	49 391
Total cost	$83 584

On December 20th and 21st, 1905, the writer conducted evaporative tests, the data and result of which are given in Table 17. Each boiler, with its furnace, was tested separately, under similar conditions.

Figs. 1 and 2, Plate XXXIV, show the average condition of the smoke during the tests.

Fig. 10 shows an average steam card produced on a night run.

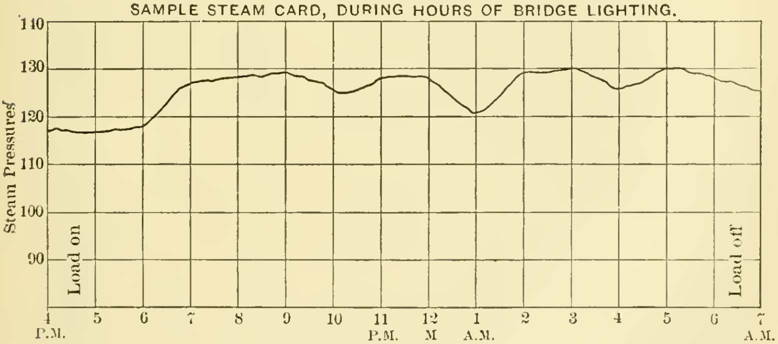


FIG. 10.

During the ordinary weekday, the quantity of rubbish consumed is about 160 loads, equivalent to 22 560 lb.

TABLE 17.—DATA AND RESULTS OF EVAPORATIVE TESTS; RUBBISH INCINERATOR AND ELECTRIC LIGHTING STATION, DELANCEY SLIP, BOROUGH OF MANHATTAN, NEW YORK.

Trials made by H. de B. Parsons.

Kind of fuel, city rubbish collections.

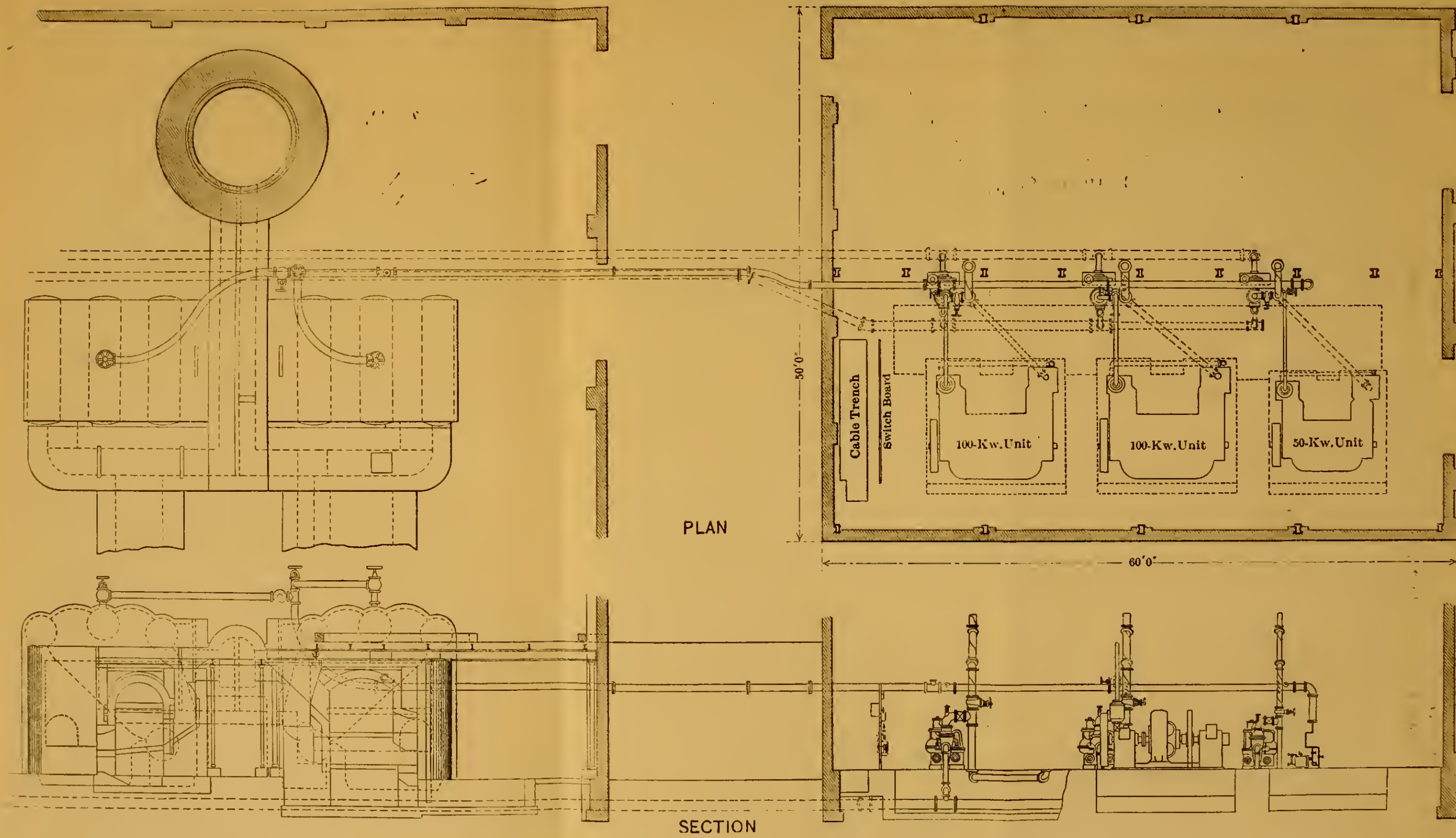
Method of starting and stopping, alternate.

Make of boilers, Stirling Water Tubular.

Data.	West boiler.		East boiler.	
Grate surface of furnace		113 sq. ft.		74 sq. ft.
Effective water-heating surface		1 890 " "		1 890 " "
Surface of feed-water heater coil in flue		60 " "		60 " "
TOTAL QUANTITIES.				
Date of trial		Dec. 20th, '05.		Dec. 21st, '05.
Duration of trial		5.5 hours.		5.5 hours.
Weather		Fair.		Rainy.
Condition of rubbish		Dry.		Wet.
Weight of rubbish delivered		31 193 lb.		21 175 lb.
Weight of rubbish picked out as: marketable		8 926 "		7 245 "
paper	6 876		6 435	
rags	1 800		610	
cans	250		200	
Weight of rubbish burned		22 267 "		13 930 "
Weight of ash, estimated		10%		10%
Total weight of water fed to boiler		29 925 "		24 675 "
Equivalent water evaporated, from and at 212°		36 568 "		30 054 "
Number of furnace men:		9		4
stokers	6		2	
feeders	3		2	
Equivalent evaporation per man per ton		365 "		1 078 "
HOURLY QUANTITIES.				
Rubbish consumed per hour		4 048.5 "		2 532.7 "
" " " " per square foot of grate		35.8 "		34.2 "
Water evaporated per hour		5 440.9 "		4 486.2 "
Equivalent evaporation per hour, from and at 212°		6 648.7 "		5 464.2 "
Equivalent evaporation per hour, from and at 212°, per square foot of heating surface		3.51 "		2.89 "
AVERAGES.				
Temperature of external air		46° fahr.		49° fahr.
Barometer, inches		30.32		29.64
Steam pressure by gauge		117 lb.		100 lb.
Temperature of feed-water		40° fahr.		40° fahr.
Temperature of gases entering boiler		....		1 740° "
" " " under "		1 525° fahr.		1 400° "
" " " escaping from boiler		466° "		412° "
Force of draft, under boiler, inches		0.75		0.55
" " " in flue to stack, inches		1.30		1.15
Boiler horse-power developed		192.7		158.4
ECONOMIC RESULTS.				
Water evaporated, actual, per pound of rubbish		1.34 lb.		1.77 lb.
Equivalent evaporated per pound of rubbish		1.64 "		2.16 "

The cost of operating the plant by the City, showing the debits and credits, is given in the financial statement, Table 18. Prior to the construction of the plant, the material was loaded on scows and taken to land-fills. (Previous to the administration of Commis-

PLAN AND SECTION THROUGH ELECTRIC LIGHTING STATION.



sioner Woodbury, much of this material was dumped at sea.) These scow loads contained the collections of ashes, street-sweepings and rubbish. The rubbish, therefore, was reduced in bulk, both by compression and by the ashes filling in the voids.

The cost to the City, for towing, unloading and scow hire, averaged, in 1905, $0.1569 per cu. yd. of mixed material. From a study of the records kept by the City, the reduction of the rubbish as collected and delivered occupies three-tenths of its original volume when loaded on the scows.

An average day's work at the plant represents about 1 050 cu. yd. of rubbish delivered during 24 hours.

TABLE 18.—Financial Statement.

One Day's Work. Delancey Slip Plant.

Cost of disposal of rubbish on land-fills.		
1 050 cu. yd. delivered, compressed on scows, after trimming, to 315 cu. yd.		
315 cu. yd. at $0.1569		$49.42
Cost, Incinerator.		
Labor	....	
Ash removal, 7.6 cu. yd. at $0.1569	$1.20	
Supplies and repairs	8.00	
Interest, 3½% on $34 193	3.28	
		12.48
Saving per day		$36.94
Saving per year, $13 483, or 39.4% on cost.		
Cost, Electric Lighting Station.		
Cost of buying electricity		$80.00
Labor	$20.00	
Supplies, repairs and sundries	8.00	
Interest, 3½% on $49 391	4.74	
		32.74
Saving per day		$47.26
Saving per year, $17 250, or 35% on cost.		

The total saving, as shown by Table 18, on the combined plant is \$30 733 per annum, or 36.7% on the cost.

There is included in Table 18 no cost for labor charges in the incinerator plant, because the privilege of picking out the marketable rubbish on the belt conveyor is under contract, and the contractor pays to the City a sum which slightly exceeds the expenses of labor in the incinerator building, including the operation of the boilers. Taxes are not included, because the plant is built on City property, purchased to protect the bridge structure overhead.

Prior to the erection of the Delancey Slip plant, the Department of Street Cleaning made tests on the burning of rubbish in a building on North Moore Street and at the rubbish incinerating plant erected at the foot of West Fourth Street, with the results given in Table 19. These experiments were carried on during 1903 and 1904. At the North Moore Street plant a second-hand return-tubular boiler was used, with a grate and setting arranged to accommodate the rubbish.

TABLE 19.

	North Moore St.	Forty-seventh Street Incinerator.	
Duration of trial, hours	6	2	3
Grate surface, square feet	....	90	90
Boiler heating surface, square feet	324	2 760	2 760
Rubbish burned, pounds	3 324	9 316	10 054
Temperature of feed water, degrees, fahr.	56°	50°	50°
Steam pressure, pounds	10	80	80
Water evaporated, pounds	3 968.75	11 101.00	....
Equivalent evaporation, from and at 212° fahr., pounds	4 648	13 365	15 139
Equivalent evaporation, from and at 212° fahr., pounds per pound of rubbish	1.40	1.43	1.50
Temperature of furnace, degrees, fahr.	1 500°	....	....
Ashes removed, pounds	....	907	....
British thermal units required for evaporation of water from and at 212° fahr., per pound of rubbish	1 352.4	1 381.4	1 449.0

The ashes produced at the Forty-seventh Street and at the Delancey Slip rubbish incinerating plants were analyzed by the Lederle Laboratories, with the following results:

Sample of Ashes from West Forty-seventh Street Incinerator:

Moisture	2.12%
Potassium carbonate	2.65%
Calcium phosphate	1.98%
Alkaline earth carbonates, silicates, soda, oxides of iron and alumina, etc.	68.05%
Organic and volatile matter (loss on ignition)	25.20%
	100.00%

Sample of Ashes from Delancey Slip Incinerator:

Moisture	0.75%
Nails and other metal	5.48%
Broken glass	4.05%
Bone phosphate	2.71%
Potash	0.46%
Alkaline earth carbonates, silicates, soda, oxides of iron and alumina, etc.	60.91%
Organic and volatile matter (loss on ignition)	25.64%
	100.00%

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

CONCERNING THE INVESTIGATION OF OVERLOADED BRIDGES.

By Wilbur J. Watson, M. Am. Soc. C. E.

To be Presented September 5th, 1906.

For some years a large part of the writer's professional work has been the investigation of existing bridges which are subjected to loads greater than those for which they were originally designed; and, in the course of this work, he has found, among bridge engineers, such a divergence of views regarding the maximum unit stresses, etc., which may be safely allowed in an existing structure, that he takes this opportunity of bringing the matter before the Society for discussion, in the hope that thereby greater uniformity of opinion may be established. The rapid development of electric traction lines, both urban and interurban, many of which are built upon highways, has had the effect of overloading many highway bridges.

The writer has in mind an interurban line which was built upon a highway and crossed many bridges. The engineer in charge, who was not a bridge engineer, reviewed the stresses caused by the new loading, and condemned every structure in which they exceeded those recommended by a certain standard specification for highway

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

bridges. On the other hand, lines have been built over old highway bridges without any investigation whatever being made as to the strength of the bridges. It is hardly necessary to state that such lines were built by promoters, without the aid of competent engineers.

One promoter, desiring to show his contempt for engineers in general, told the writer that he had built an interurban line, ten miles in length, and that his total expense for engineering services amounted to $75. The writer having ridden over this line, sees no reason to doubt the promoter's statement.

It is the writer's practice, when called upon to report concerning the safety of an existing structure, to make a careful examination of the structure in the field, in order to determine the following points:

First.—The general dimensions;

Second.—The sizes of the members, connections, etc., making full allowance for corrosion, if any;

Third.—The character of the material, whether steel or iron;

Fourth.—The workmanship;

Fifth.—The presence of, or liability to, secondary stresses.

The stresses are then computed, and to the dead-load and live-load stresses an amount is added to cover those due to impact, vibration and similar secondary stresses. In computing this allowance for secondary stresses, the rational impact formula, $I = L \dfrac{L}{L + D}$, is used, in which I equals the allowance to be made for secondary stresses, L, the stress due to the live load considered as a static load, and D, the stress due to the dead load only. In case there exist secondary stresses due to eccentric connections, which are very common in old highway bridges, or due to other defects in design or execution, these should be added to obtain the maximum.

In regard to allowable unit stresses, it may be stated, as a general principle, that the greatest possible stress in any member should not exceed the elastic limit of the material.

There are two reasons why this limit has been used:

First.—In exceeding the elastic limit, the stress causes a permanent distortion of the member, which, in turn, causes a change in the conditions which were assumed in computing the stresses.

Second.—The classical experiments of Wöhler and Bauschinger

demonstrated that repeated strains greater than the elastic limit of the material, but much less than the ultimate strength, might cause rupture.*

It is well known that eye-bars and built-up members do not develop the full strength indicated by their sections and by tests on small-sized pieces, and it would appear that it is not safe to assume the elastic limit of full-sized members to be more than about 75% of the elastic limit indicated by tests on small test pieces of the material of which these members are composed. It has also been demonstrated that single angles, connected by one leg only, cannot be depended upon to develop more than about 60% of their computed strength.†

The second cardinal principle, then, as used by the writer, may be stated as follows: The greatest computed unit stress in any member in tension, making full allowance for ordinary and extraordinary secondary stresses, should never exceed 75% of the elastic limit of the material for members symmetrically connected, nor 60% of the elastic limit of the material for angles connected by one leg only. The latter case is very often found in old lattice trusses. This would give, for symmetrically connected members in tension, working stresses of, approximately, 20 000 lb. per sq. in. for iron, 24 000 lb. per sq. in. for soft steel, and 26 000 lb. per sq. in. for medium steel.

In examining old bridges, it is necessary to note carefully the construction of the compression members, as they are often insufficiently stiffened, especially the T-sections formerly used to such an extent. The writer will risk the statement, drawn entirely from personal knowledge, that more failures occur in compression members than in tension members, and he has in mind the failure of an electric railway bridge by the buckling of the top chord (a T-section) under a unit stress in the steel not exceeding 22 000 lb. per sq. in. Full allowance for impact, etc., as outlined above, had been made, and the failure was entirely due to the lack of proper stiffening of the member.

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* Paper by H. B. Seaman, M. Am. Soc. C. E., on "The Launhardt Formula, and Railroad Bridge Specifications," and discussion thereon by C. C. Schneider, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XLI, pp. 140 and 173.

† Paper by J. E. Greiner, M. Am. Soc. C. E., on "Tests of Bridge Members," *Transactions*, Am. Soc. C. E., Vol. XXXVIII, p. 41.

TABLE 1.—STRESSES IN STRINGERS AND FLOOR-BEAMS OF RAILWAY BRIDGES.

Full allowance has been made for impact, etc., by the use of the impact formula, $I = L\,\dfrac{L}{L + D}$.

All these stresses were obtained from the wheel-loads actually used.

No.	Railroad.	Member.	No. of members.	Flange unit stress.	Maximum rivet-bearing on web.	Material.	Examined in :	Replaced in :	Remarks.
1	Railroad	Stringers	....	18 300	45 000	Iron.	1901	....	In use, 1905.
2*	....	"	32	10 200	39 000	"	"	....	" "
3*	....	"	4	9 500	52 400	"	"	....	" "
4	....	"	8	19 100	49 100	"	"	....	" "
5	....	"	....	15 500	38 000	Steel.	1900	1903–04	
6	....	"	....	27 900	31 000	"	"	"	
7	....	Deck-span	....	16 700	28 800	Iron.	"	"	
8	....	"	....	21 400	35 600	"	"	"	
9	....	"	....	23 500	37 000	"	"	"	
10	....	Floor-beams	....	16 000	38 000	"	"	"	
11	Traction Co.	"	....	23 300	33 100	Steel.	1903	....	In use, 1905.
12	"	"	....	23 050	33 300	"	"	....	" "
13	Railroad	Stringers	....	18 700	37 700	Iron.	1902	....	Not known, if in use now.
14	"	"	....	25 600	45 800	"	"	....	"
15	"	Floor-beams	....	20 600	43 000	"	"	....	"
16	"	Stringers	12	23 200	42 400	Steel.	1900	1904–05	
17	"	Floor-beams	5	21 000	35 700	"	"	"	
18	"	Stringers	....	19 520	36 700	"	"	"	
19	" Trunk line	"	....	29 800	52 300	"	1900	"	
20	"	Stringers and floor-beams	70	20 000 (av.)	40 000 (av.)	Iron.	"	....	Still in use, 1905.

* Nos. 2 and 3 were old stringers cut off and used as deck spans, which accounts for the great disproportion between the flange unit stresses and the bearing stresses of the flange rivets.

It is the writer's practice to increase the stress in compression members by the following formula, $S_2 = S_1 \left(1 + \frac{l^2}{18\,000\, r^2}\right)$, in which S_1 = the direct unit stress, S_2 = the maximum unit stress, l = the length of the member, and r = the radius of gyration of the member, in inches. This will be recognized as Gordon's formula as ordinarily used for pin-connected columns.

For computing the stresses in an improperly designed column, no general method is offered, as it is governed by the peculiarities of each case. It is in cases such as these that good engineering or structural judgment is most needed.

Many structures are unnecessarily condemned on account of rivets overstressed in shearing and bearing. The usual assumptions governing such cases are as follows:

First.—That the shearing strength of the material is equal to two-thirds of the tensile strength, and that the bearing strength is equal to twice the shearing strength;

Second.—That the entire stress transferred from one member to another must be considered as transferred entirely by the rivets in shearing and bearing, making no allowance whatever for the friction between the surfaces of the members;

Third.—That the diameter of the rivet is assumed as the diameter before driving.

These assumptions would give maximum allowable stresses for iron rivets of, approximately, 13 300 lb. per sq. in. in shear, and 26 000 lb. per sq. in. in bearing. The writer's experience has convinced him that riveted connections will not fail under any such limits of stress, and, in general, rivets will not begin to work loose until the unit stresses are nearly twice as great as these.

Table 1 gives some of the unit stresses in bearing of the flange rivets of built-up stringers of railway bridges examined or computed by the writer. All these bridges were on railroads carrying heavy traffic, and, in computing the stresses, full allowance was made for impact, etc. Not one of these stringers, as far as the writer is aware, ever showed signs of failure. The tensile flange unit stress is also shown, by way of comparison with the bearing stress.

The writer has considered that the reason for this great discrepancy between theory and observed results lies in the neglect of the element of friction between the two faces of the members in contact. This theory seems to be confirmed by the results of such tests as have been made.

Tests to determine the friction of riveted joints were made at the Watertown Arsenal, in 1882. These demonstrated that the friction developed between the surfaces of the plates of a riveted joint is a function of the area of the rivet section, and, therefore, is directly comparable with the shearing value of the rivet. These tests show the value of the friction to be about 14 000 lb. per sq. in. of rivet section for the single surfaces in contact, for a lap-joint, and about 18 000 lb. per sq. in., for joints providing two contact surfaces. As these values are higher than those usually allowed for rivets in shear, it follows that, in riveted joints, the working stresses are transmitted entirely by friction, and the shearing and bearing resistances of the rivets are not brought into play until much higher values of stress are reached.

It would appear that the ordinary working values of rivets in shear and bearing might well be increased, in the design of new work, and that the maximum allowable stresses in old structures might also be increased.

European bridge engineers have stated that European practice allows a much higher value of rivets in shear and bearing than American practice, and that they have never had trouble from this source. Therefore, it is perfectly safe to allow shearing and bearing values equal to the allowed value of the material in tension for shear, and, to double this amount for bearing, provided there are enough rivets in the connection, and that the workmanship is such as required to develop friction between the surfaces of the material. This would give values of, approximately, 22 500 lb. per sq. in. in shear, and 45 000 lb. per sq. in. in bearing, for rivet steel having a minimum elastic limit of 30 000 lb. per sq. in., to be used in examining old structures.

A set of experiments on riveted joints was recently made for a committee of the American Railway Engineering and Maintenance-of-Way Association.* These experiments were made on ninety

* Bulletin No. 62, American Railway Engineering and Maintenance-of-Way Association, April, 1905.

small riveted test pieces, which were tested to destruction, the failure occurring, in a majority of cases, by the shearing of the rivets at unit stresses of from 45 000 to 50 000 lb. per sq. in. of rivet section before driving, or by the breaking or splitting of the plates.

The values of the rivets in bearing upon the plates were approximately twice the shearing values at the point of failure. The conclusions drawn by the committee are repeated here:

"(1) That the resistance of a riveted joint against deformation by shearing forces, up to the yield point, is due to the friction between the surfaces held in contact by the rivets.

"(2) That the yield point of a riveted joint is reached when the shearing forces are equal to the friction of the surfaces held in contact by the rivets.

"(3) That the deformation of a riveted joint at the yield point is caused by the slipping on each other of the surfaces held in contact by the rivets, and is due to the diametral contraction of the rivets in cooling after they are driven, which leaves a space between the body of the rivet and the edge of the rivet hole.

"(4) That after the slip at the yield point has occurred and the rivet is brought to bear against the edge of the rivet hole, a deformation of the body of the rivet takes place with an accelerating increase in the resistance until the entire side of the rivet has been brought to bear against the edge of the rivet hole, and that the deformation continues beyond this point with a diminishing increase in the resistance until the ultimate strength of the rivet in shear has been reached and the breakdown occurs.

"(5) That lap joints, on account of the unsymmetrical distribution of material, deflect sideways under strain, throwing the rivets in tension, and thereby reducing the shearing forces between the surfaces held in contact by the rivets.

"(6) That fillers inserted between the main plates reduce the strength of a riveted joint, but that the full strength can be obtained by connecting the fillers to the main plates by additional rivets.

"(7) That the number of rivets connecting the fillers to the main plates should, for each intervening filler, be about one-third the number of rivets required in a similar joint without fillers, to obtain the same strength in both cases.

"(8) That the strength of a riveted joint with rivets of larger grip than about four times their diameter is decreased, as the length of the grip is increased.

"(9) That the number of rivets, in a riveted joint with larger grip of the rivets than four times their diameter, should be increased

at least one per cent. for each one-sixteenth of an inch increase in the grip above this length, to obtain the same strength as a similar joint with the grip of the rivets shorter than four times their diameter.

"(10) That a riveted joint, subject to forces always acting in the same direction, may safely be strained beyond the yield point up to a point where the rivets are brought to bear against the edges of the rivet holes.

"(11) That a riveted joint subject to forces alternating in opposite directions, may not safely be strained up to the yield point.

"(12) That, to obtain a minimum slip at the yield point, it is necessary that the holes in the component pieces should thoroughly match and that the driving tool should upset the rivet throughout its length so that it will thoroughly fill the rivet hole."

If the third conclusion be true, that rivets, in cooling, contract to such an extent that a space is left between the rivet surface and the side of the rivet hole, why is it that all rivets do not test loose under the inspector's hammer? It is the firm conviction of experienced inspectors that they can detect any rivet that is not completely upset. Is it possible that these men are mistaken, and that rivets which test loose do so only because the grip of the rivet heads on the plate is not great enough to prevent their movement?

This theory has been advanced before by European engineers, but the writer has seen specimens of riveted joints, which, when sawn through the rivet and polished, showed only a fine hair line dividing the rivet surface from the sides of the hole, which indicated a perfectly tight rivet. Of course, as is well known, in the case of long rivets, they are entirely upset only at the ends, and a space is often left between the rivet surface and the side of the hole at the center of the shank. Furthermore, the writer is loath to believe that the many thousands of rivets which he has tested and found to ring true could have done so unless the rivet sides were in contact with the sides of the hole, at least for some distance under the head.

Cannot the slipping of the plates, at the stress which is sufficient to overcome the friction between their surfaces, be explained on the ground of the compressibility of the metal, since apparently the slipping always occurs at a high stress. The experiments of M. Considère, in France, indicate slipping at a stress equal to ap-

proximately four-tenths of the ultimate shearing strength of the rivets, while the records of the experiments, conducted by the committee referred to, do not indicate any extraordinary movement of the plates when the slip occurred, subsequent increments of load causing movements of the plate comparable to the original slip. Is it not probable that the driving operation causes a state of compression to exist in the rivet metal and in the surrounding plate sufficient to take up the contraction due to cooling?

When the stress becomes sufficient to overcome the friction between the plates, the sudden reduction of the coefficient of friction would bring about half this stress suddenly upon the rivets, and considerable movement would be expected, due to the compression of the material, even if the rivets were perfectly tight.

There is another point, which concerns the design of new work as well as the investigation of old work, to which attention should be called.

In the design of plate girders it is the general custom, in the United States, to take the effective depth of the girder as the distance between the centers of gravity of the flanges, when obtaining flange stresses, and the distance between the rivet lines as the effective depth when obtaining the rivet values, which are dependent upon the stress transferred from the webs into the flanges in a given distance. Since the latter stress is simply an increment of the former, it would seem inconsistent to use different depths of girder for the two cases. Would not the more rational method be to assume the effective depth of the girder as variable, and use the same depth for both purposes?

At the ends of the girder there is no flange stress, and, therefore, the effective depth of the girder is evidently the distance from center to center of rivets, while at the center of the girder, considering the same to be uniformly loaded, there is no shear, and all the stress has found its way into the flanges. Evidently, then, the distance between the gravity lines of the flanges is the correct depth of the girder at this point, if the stress be uniformly distributed throughout the flange section, as it seems reasonable to assume.

The writer has found many cases of existing girders in which the maximum rivet stresses were found, not at the ends of the girder, but at an intermediate point; and he would recommend that,

in such cases, a depth intermediate between the distance from center to center of gravity of the flanges, and the distance from center to center of the rivet lines, be taken as the effective depth for all purposes.

There is another point which illustrates well the difference between rules for designing new structures, and those for investigating old ones.

It is common practice, in designing plate girders, to allow no part of the web to be considered as flange section. While this gives an added factor of safety to the structure, and may be commendable in designing new work, there can be no doubt but that the web will do its share of the work involved in resisting flange stresses, in spite of the prohibition, and this fact should be fully considered in determining the strength of an existing structure, as, in such cases, it is desirable to know the actual conditions of stress.

The writer is a firm believer in the use of standard specifications as a guide in designing standard structures, and believes just as firmly that they should not be followed in designing structures which are of an extraordinary or unusual nature, and should not be used when reporting upon the safety of a structure, which is, in itself, unusual, or is subjected to unusual conditions of loading.

The general principles of bridge design should constitute the engineer's sole guide in such cases, and his only purpose should be to determine the actual conditions of stress in the various parts of the structure, basing his computations on actual measurements, and "rock-bottom" principles, unhampered by the limitations and assumptions of a set of specifications gotten up for an entirely different purpose.

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PAPERS AND DISCUSSIONS.

THE ECONOMICAL DESIGN OF REINFORCED CONCRETE FLOOR SYSTEMS FOR FIRE-RESISTING STRUCTURES.

Discussion.*

By Messrs. H. T. Forchhammer, Arthur W. French, Irving P. Church, B. R. Leffler and George Hill.

Mr. Forchhammer. H. T. Forchhammer, Assoc. M. Am. Soc. C. E.—Without doubt, reinforced concrete will be used instead of steel for many and various constructions, in the near future, and any contribution to the theory or practical design is warmly welcomed.

The speaker thinks that Captain Sewell has done right in placing the economical design in the foreground, but does not quite approve the way in which the problem has been solved.

The stress-strain curve, as advised by the author, is probably the best one available until the results of experiments—made especially for this purpose—are at hand.

In opposition to the author, the writer believes that total failure occurs when the stress in the steel reaches the elastic limit. Hence, the ultimate strength of the beam is reached when the steel is stressed to the elastic limit, or the concrete is compressed to its ultimate strength.

The considerations for economical design, which will be given herewith, will show that it is not always the cheapest beam which is designed for these two values simultaneously.

* Continued from March, 1906, *Proceedings*. See December, 1905, *Proceedings* for paper on this subject by John S. Sewell, M. Am. Soc. C. E.

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The author advises the use of Equation 6 instead of Equations 1 to 5. If, in Equation 6, a is replaced by the proportion of d given on page 634,* this equation will give practically the same results as Equations 1 to 5, but, if a and d are considered as variables, as on page 637,* the connection between Equations 6 or 7 and Equations 1 to 5 is quite changed.

Suppose Equation 7 to be solved for d, using a value for a which is smaller than the one given on page 634;* the result is a beam in which the concrete is stressed less than 2 000 lb. per sq. in. when the steel is stressed to its elastic limit. If—on the other hand—Equation 7 is solved for d, using for a a value larger than given on page 634,* the result is a beam in which the concrete is stressed more than 2 000 lb. per sq. in. when the steel is stressed to its elastic limit.

The result of the author's economical considerations, therefore, is that, for concrete beams with reinforcement of low elastic limit, it is cheaper to proportion the beam with less steel than that given on page 634,* but the author says nothing about the relative cost of beams reinforced with various kinds of steel. This can very well be done, and the speaker disagrees absolutely with the author when, on page 636,* he says it is impossible—with Equations 1 to 5—to express the cost in terms of the ratio of cost of steel per cubic foot to the cost of concrete per cubic foot, because this ratio is entirely independent of the ratio between the maximum allowable stresses of the two materials. Just because these two ratios are entirely independent of each other it is possible to find a value for the last-named ratio, leading to minimum cost. To accomplish this the speaker will use Equations 1 to 4, and, instead of Equation 5, will use Equation 6.

With $b = 1$, $\frac{E_s}{E_c} = e$.

Let $F =$ the maximum compression in concrete.

Using Equations 3 and 1:

$$\frac{y_1}{1} = \frac{y_2}{\frac{t_s}{e\,F}} = \frac{d}{1 + \frac{t_s}{e\,F}}.$$

Substituting β for $1 + \frac{t_s}{e\,F}$, and solving:

$$y_1 = \frac{d}{\beta},$$

$$y_2 = d\left(1 - \frac{1}{\beta}\right).$$

* *Proceedings*, Am. Soc. C. E., for December, 1905.

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This, substituted in Equation 4, gives

$a\,t_s = 0.57\;F \times \frac{d}{\beta}$, which, substituted in Equation 6, gives

$$m = h \times d \times 0.57\;F \times \frac{d}{\beta}.$$

Substituting α for $\frac{0.57\;F\,h}{\beta}$, and solving:

$$d = \sqrt{\frac{m}{\alpha}}.$$

To express h by t_s, Fig. 1 gives

$$h\,d = y_2 + n\,y_1,$$

$$h = 1 - \frac{1-n}{\beta},$$

for $$n = 0.64, \quad h = 1 - \frac{0.36}{\beta}.$$

Substituting the value of a in Equation 9,

$$x = p \times 0.57\;F \times \frac{d}{\beta\,t_s} + d,$$

as $$0.57\;F \times \frac{h}{\beta} = \alpha,$$

and $$d = \sqrt{\frac{m}{\alpha}},$$

$$\frac{x}{\sqrt{m}} = \frac{1}{\sqrt{\alpha}} + \frac{\sqrt{\alpha}\,p}{h\,t_s},$$

as $$\beta = 1 + \frac{t_s}{e\,F},$$

$$h = 1 - \frac{0.36}{\beta}$$

$$\alpha = 0.57\;F \times \frac{h}{\beta},$$

it will be seen that x is expressed in terms of t_s, e, F, and p.

As e and p are constants, x is a function of F and t_s only; hence, for each value of t_s, there is a certain value of F, making x a minimum, or, for each value of F, there is a certain value of t_s, making x a minimum.

These values, naturally, are purely theoretical, and may be outside the practical existing limits; but as the special object is not to get an absolute minimum, but only a good economical design, the speaker is of the opinion that it may be of interest to find values for the relative cost of beams designed for various values of t_s and F.

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In the formula just developed,

$$\frac{x}{\sqrt{m}} = \frac{1}{\sqrt{\alpha}} + \frac{\sqrt{\alpha}\, p}{h\, t_s},$$

the first member, $\frac{1}{\sqrt{\alpha}} = \frac{d}{\sqrt{m}}$, is due to the cost of the concrete; the last member, $\frac{\sqrt{\alpha}\, p}{h\, t_s}$, is due to the cost of the steel. In Tables 3 to 7, C has been substituted for $\frac{1}{\sqrt{\alpha}}$, and S for $\frac{p\sqrt{\alpha}}{h\, t_s}$. In these tables, in accordance with the author's opinion, p is assumed to be 72, and e to be 15. The ultimate strength of concrete in compression is assumed to be 2 300 lb. per sq. in. The upper part of Table 3 is for medium steel, with an elastic limit of 33 000 lb. per sq. in. For the various percentages of steel, the speaker has calculated the compression in the concrete, represented by F; the depth of the beam, by $\frac{d}{\sqrt{m}}$; the cost of the concrete, by C; the cost of the steel, by S; the total cost, by $\frac{x}{\sqrt{m}}$; and the resisting moment is for a beam 1 in. wide and 12 in. deep.

It will be seen that the cheapest beams are those in which the percentage of steel is from 1.1 to 1.4, and that the cost does not vary much if the percentage is only a little outside of these limits.

The lower part of Table 3 is for hard steel, with an elastic limit of 57 000 lb. per sq. in. For the other items the values are the same as in the upper part of Table 3.

It will be seen that the cheapest beam is that in which the full strength of the concrete and the elastic limit of the steel are reached simultaneously. Here, also, a little variation in the percentage of the steel does not influence the cost very much; but, as there is no actual minimum value (but two independent curves intersecting at the minimum point), it is here more important to keep close to the right percentage.

To try these results on tests the speaker used a very interesting series of tests* made under the supervision of A. N. Talbot, M. Am. Soc. C. E.

The upper part of Table 4 is for 12-in. beams of medium steel (all in the series) with an elastic limit varying from 30 000 to 35 000 lb. per sq. in. This table shows the actual percentage of

* The results of these tests are published in Bulletin No. 1 of the University of Illinois Engineering Experiment Station, September 1st, 1904.

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steel; the final stress in the steel; the breaking moment, in inch-pounds per inch; and the cost,

$$\frac{x}{\sqrt{m}} = \frac{12}{\sqrt{m}}\left(1 + 0.72 \times \frac{100\,a}{d}\right).$$

From Table 3 is taken the breaking moment and cost, as calculated, assuming the elastic limit to be 33 000 lb. per sq. in.

TABLE 3.—COST OF REINFORCED CONCRETE BEAMS, WITH VARIOUS PERCENTAGES OF REINFORCEMENT.

MEDIUM STEEL.

$100\frac{a}{d}$.	t_s.	F.	β.	h.	a.	Cost Data. $C = \frac{d}{\sqrt{m}}$	S.	$\frac{x}{\sqrt{m}}$	For $d = 12$ in., m.	Remarks.
0.41	33 000	850	3.59	0.900	122	0.0906	0.0267	0.1173	17 600	
0.52	33 000	980	3.25	0.889	153	0.0809	0.0303	0.1112	22 000	
0.83	32 000	1 300	2.70	0.867	238	0.0649	0.0388	0.1037	34 200	
1.11	33 000	1 560	2.42	0.851	312	0.0567	0.0455	0.1022	44 900	
1.39	33 000	1 800	2.23	0.838	386	0.0510	0.0510	0.1020	55 300	Minimum cost.
1.56	33 000	1 940	2.14	0.832	429	0.0483	0.0543	0.1026	61 700	
2.03	33 000	2 300	1.96	0.816	546	0.0429	0.0625	0.1054	78 600	

HARD STEEL.

$100\frac{a}{d}$.	t_s.	F.	β.	h.	a.	$C = \frac{d}{\sqrt{m}}$	S.	$\frac{x}{\sqrt{m}}$	For $d = 12$ in., m.	Remarks.
0.42	57 000	1 490	3.55	0.899	216	0.0681	0.0206	0.0887	31 000	
0.70	57 000	2 020	2.88	0.875	349	0.0536	0.0270	0.0806	50 200	
0.87	57 000	2 300	2.65	0.864	427	0.0484	0.0302	0.0786	61 500	Minimum cost.
0.97	53 200	2 300	2.54	0.858	442	0.0476	0.0332	0.0808	63 800	
1.52	39 800	2 300	2.16	0 833	505	0.0445	0.0486	0.0931	72 600	

With one exception (Beam No. 9) it will be seen that the final stress in the steel varies between the limits 29 300 and 37 400, with an average of 32 900 lb. per sq. in. With the same exception there is also harmony between the ultimate moments and costs as they were in the actual beams and as they are calculated. It would seem that the calculated moments are somewhat too small for the small percentages and somewhat too large for the large percentages.

The lower part of Table 4 is for six beams of hard steel having an elastic limit of from 55 000 to 60 000 lb. per sq. in. In this table, also, there is evidence of harmony between theory and practice.

To complete the investigation, Table 5 shows the relative cost of beams reinforced with steel of various elastic limits.

Assuming the ultimate strength of concrete in compression to be 2 300 lb. per sq. in., it will be seen that the cost decreases with the elastic limit of the steel until this has reached about 100 000 lb. per sq. in. Any further increase in the elastic limit will not decrease the cost.

TABLE 4.—COST OF 12-IN. REINFORCED CONCRETE BEAMS WITH VARIOUS PERCENTAGES OF REINFORCEMENT. Mr. Forchhammer.

MEDIUM STEEL.

Beam No.	Reinforcement in 12 × 12-in. beam.	Percentage of Steel.	Final stress in steel measured.	Breaking Moment, m, per 1 in. width of beam. Actual.	Breaking Moment, Calculated from Table 3.	Total Cost, $\frac{x}{\sqrt{m}}$. Actual.	Total Cost, From Table 3.
19	3 ½-in. plain round	0.41	37 400	21 400	17 600	0.106	0.117
21	3 ½-in. " "	0.41	32 900	18 700	17 600	0.113	0.117
9	3 ½-in. Ransome	0.52	70 000	42 000	22 000	0.081	0.111
16	3 ½-in. plain square	0.52	32 100	23 100	22 000	0.108	0.111
17	3 ½-in. " "	0.52	29 300	22 200	22 000	0.110	0.111
5	3 ½-in. Kahn	0.83	30 600	30 400	34 200	0.110	0.104
10	3 ¾-in. Thacher	0.83	32 000	33 800	34 200	0.104	0.104
15	3 ¾-in. "	0.83	35 000	36 200	34 200	0.101	0.104
14	4 ½-in. Kahn	1.11	30 200	39 700	44 900	0.108	0.102
4	5 ½-in. "	1.39	33 800	49 000	55 300	0.108	0.102
27	4 ¾-in. plain square	1.56	34 900	58 300	61 700	0.105	0.103
22	3 ¾-in. Kahn	1.67	33 800	51 400	66 000	0.116	0.103

HARD STEEL.

Beam No.	Reinforcement in 12 × 12-in. beam.	Percentage of Steel.	Final stress in steel measured.	Breaking Moment, Actual.	Breaking Moment, Calculated from Table 3.	Total Cost, Actual.	Total Cost, From Table 3.
3	3 ½-in. Johnson	0.42	52 600	28 000	31 000	0.095	0.089
7	3 ½-in. "	0.42	58 300	30 300	31 000	0.091	0.089
2	5 ½-in. "	0.70	58 800	44 300	50 200	0.086	0.081
20	5 ½-in. "	0.70	65 200	46 600	50 200	0.083	0.081
13	7 ½-in. "	0.97	54 100	64 100	63 800	0.080	0.081
28	6 ¾-in. "	1.52	50 300	72 400	72 600	0.093	0.093

TABLE 5.—COST OF BEAMS REINFORCED WITH STEEL OF VARIOUS ELASTIC LIMITS.

t_s.	F.	$100\frac{a}{d}$.	β.	h.	a.	Cost-Data. $C = \frac{d}{\sqrt{m}}$.	Cost-Data. S.	Cost-Data. $\frac{x}{\sqrt{m}}$.	For d = 12 in., m.	Remarks.
30 000	1 630	1.39	2.23	0.838	348	0.0536	0.0536	0.1072	50 100	
33 000	1 800	1.39	2.23	0.838	386	0.0510	0.0510	0.1020	55 300	
35 000	1 910	1.39	2.23	0.838	407	0.0496	0.0496	0.0992	58 600	
42 300	2 300	1.39	2.23	0.838	493	0.0451	0.0451	0.0902	71 000	Minimum depth.
50 000	2 300	1.07	2.45	0.853	456	0.0469	0.0361	0.0830	65 700	
55 000	2 300	0.92	2.59	0.861	435	0.0480	0.0318	0.0798	62 700	
60 000	2 300	0.80	2.74	0.869	415	0.0491	0.0283	0.0774	59 700	
70 000	2 300	0.62	3.03	0.881	381	0.0513	0.0229	0.0742	55 000	
80 000	2 300	0.49	3.32	0.892	352	0.0534	0.0188	0.0722	50 600	
90 000	2 300	0.40	3.61	0.900	326	0.0554	0.0160	0.0714	47 030	
100 000	2 300	0.34	3.90	0.908	305	0.0573	0.0140	0.0713	44 000	Minimum cost.
120 000	2 300	0.24	4.48	0.920	269	0.0610	0.0106	0.0716	38 700	

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Naturally, this value for the elastic limit is purely theoretical, but the interesting point is, that with steel having an elastic limit of about 60 000 lb. per sq. in., practically the minimum of cost is reached, as the theoretical minimum is only 8% lower. (It must be remembered that this percentage is only part of the cost.)

Table 5 also shows that the saving effected by using hard instead of medium steel is but slight, compared with what it would be in a steel construction. Using steel with an elastic limit of 35 000 instead of 55 000 lb. per sq. in., the cost here considered will be increased by about 24%; and, assuming the part of the cost here considered to be about one-half of the total cost, it is only 12% cheaper to use the hard steel.

As medium steel is considered much safer than hard steel, for all structural purposes, especially where it is subject to impact, the speaker thinks it proper to consider the question whether the slight saving in cost really justifies the use of hard steel.

Another advantage in using medium steel is that the concrete is stressed less, hence the working stresses in the tension and also in the compression parts of the concrete are smaller; but, if medium steel is used, only very low working stresses should be allowed.

If medium steel, with an elastic limit of 35 000 lb. per sq. in., is used in a concrete having an ultimate strength of 2 300 lb. per sq. in., and a factor of safety of 4 is wanted, the working stress in the steel must be only, say, 9 000 lb. per sq. in., and in the concrete, say, 600 lb. per sq. in. As shown in Tables 3 and 5, it will be cheaper to use in the concrete a working stress of from 450 to 500 lb. per sq. in. with the 9 000 lb. per sq. in. in the steel.

In relation to T-beams, the speaker is in agreement with Captain Sewell. As steel having a low elastic limit does not utilize the full strength of the concrete in compression, even for a rectangular beam, the projecting flanges are not needed, and cannot act to decrease the dimensions of the beam. On the other hand, with steel having a high elastic limit, the full strength of the concrete is utilized in a beam of rectangular section; hence, the projecting flange will make it possible to increase the steel reinforcement without crushing the concrete. Therefore it will be proper to make $\frac{A}{b\,d} = \frac{1}{p}$, as proposed by the author, and then find h from Equation 7.

If the full width of the beam which can be considered is $B = b + 2\,c$, it is necessary to inquire whether or not $\frac{1}{p} \times \frac{b}{B}$ is greater than the percentages given in Table 5. With $p = 72$ and $\frac{B}{b} = 3$,

$$100 \times \frac{1}{72} \times \frac{1}{3} = 0.46\%,$$

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hence the equation can be used for steel having an elastic limit up to 80 000 lb. per sq. in.

For steel having an elastic limit of 60 000 lb. per sq. in. the percentage in Table 5 is 0.80. Hence, for steel having an elastic limit up to 60 000 lb. per sq. in., the equation proposed by the author may be used if $\frac{B}{b} = \frac{1.39}{0.80} = 1\frac{3}{4}$, this is, if the projecting flange on each side of the beam is at least three-eighths of the width of the beam itself.*

In the foregoing the speaker has considered bending, but has not considered shear. The shear certainly has to be taken care of, as well as the bending; but, as it is evident that the cost of a well-designed web reinforcement is entirely independent of the depth of the beam, it is proper not to consider the web reinforcement when trying to find the most economical depth.

The speaker considers the web reinforcement very important, and, without doubt, the ideal construction is with attached web members. As designed by the author, this seems to be effective and cheap.

It seems to be still an open question whether, with attached web members, it is justifiable to assume that total failure will occur before the steel is stressed appreciably beyond the elastic limit. The author explains very logically his reasons for this assumption, but the speaker doubts whether the increase in the strength of the steel, due to web reinforcement, is appreciable. It seems natural to assume that the concrete will be crushed very shortly after the stress in the steel has reached the yield point. In any event, the speaker considers it safer to assume the ultimate strength of the steel at the elastic limit until reliable experiments have shown that it is above that limit.

On page 634† the author says:

"It is certain that it can never be economical to use enough steel to cause the beam to fail first by crushing the concrete."

Table 5 shows that, for steel having a very high elastic limit, it is cheaper, and, even for steel having a lower elastic limit, it may be economical—in order to decrease the depth of the floor—to use so much steel that the beam fails first by crushing the concrete. Beam No. 13, in the lower part of Table 4, is the most economically designed of all the six beams in that series, both theoretically and practically, and it failed by the crushing of the concrete.

On page 640† the author's expression, "maximum allowable per-

* If the neutral axis is below the projecting flange—which will generally be the case—this is a little on the unsafe side; hence it will be safer to allow the 60 000 lb. per sq. in. only if $\frac{B}{b}$ is, say, 2 or greater.

† *Proceedings*, Am. Soc. C. E., for December, 1905.

Mr. Forchhammer. centage of steel," is very misleading, as it leads one to believe that additional steel is dangerous, which is not the fact. Additional steel not only decreases the stresses in the steel, but, also, the stresses in the concrete, if the moment be assumed as constant.

Quite another matter, however, is the fact that it might be fatal in solving Equation 7 for d, using t_s as the elastic limit of the steel used, when a is larger than what the author calls the "maximum allowable percentage of steel." With excessive steel, the value of t_s must be taken as that where the percentage used is allowable. An example will illustrate this: Under the assumptions made, Table 5 shows that, for steel having an elastic limit of, say, 55 000 lb. per sq. in., the most economical percentage is 0.92; then $\frac{d}{\sqrt{m}} = 0.0480$, and $\frac{x}{\sqrt{m}} = 0.0798$.

If the steel is increased to 1.07%, the allowable value for t_s is 50 000 lb. per sq. in., $\frac{d}{\sqrt{m}} = 0.0469$, or 2.3% smaller than before; $\frac{x}{\sqrt{m}} = 0.0830$, or 4.0% higher than before.

If the steel is decreased to 0.70%, Equation 7, $m = h\ a\ d\ t_s$, gives:

$$m = 0.875 \times 0.70 \times d^2 \times \frac{55\,000}{100} = 337\ d^2\ ;$$

$$\frac{d}{\sqrt{m}} = 0.0546, \text{ or } 13.8\% \text{ higher than in the first case};$$

$$\frac{x}{\sqrt{m}} = 0.0546\ (1 + 0.72 \times 0.70) = 0.0821, \text{ or } 2.9\% \text{ higher than}$$

in the first case.

It will be seen, therefore, that the most economical results are secured by using the percentages given in Table 5. Any smaller percentages will increase the cost as well as the depth of the beam. Any larger percentages will increase the cost, but decrease the depth of the beam, and the latter, under certain circumstances, may be of great value.

For these reasons, therefore, the author is incorrect in stating (page 640*) that "theoretical economy, based on relative costs, is not attainable." Theoretical economy, based on relative costs, is reached when what the author calls the maximum allowable percentages are used. The author is also incorrect in stating that any more than the economical percentage is wasted. As just shown, an

* *Proceedings*, Am. Soc. C. E., for December, 1905.

increase in the steel increases the strength of the beam; consequently, it is not wasted. Mr. Forchhammer.

With the assumption, made in Table 5, that the ultimate strength of the concrete in compression is 2 300 lb. per sq. in., the most economical percentage of steel for t_s smaller than 42 300 lb. per sq. in. is 1.4. $\frac{d}{\sqrt{m}}$ will be seen to vary approximately in proportion to t_s, according to two straight lines which intersect at a minimum point where $t_s = 42\,300$.

The following equation, proposed by the speaker, is for a rectangular beam or floor slab:

$$d = \sqrt{m}\,(\alpha + \beta\, t_s),$$

in which α and β have different values for various values of F. Assuming $F = 2\,300$, for values of t_s greater than 42 300, $\alpha = +0.0360$, and $\beta = +0.218 \times 10^{-6}$. For values of t_s smaller than 42 300, $\alpha = +0.074$, and $\beta = -0.69 \times 10^{-6}$. For other values of F, α and β will have other values, and the point of intersection of the two curves will change. To find the point of intersection for other values of F, the equation,

$$a\, t_s = 0.57\, F \frac{d}{\beta},$$

should be used, as the point of intersection is where

$$d = p\, a, \text{ or } \frac{100\, a}{d} = \frac{100}{p} = 1.39.$$

$$\frac{t_s}{F} = \frac{57}{1.39\,\beta} = \frac{41}{\beta};$$

but $$\beta = 1 + \tfrac{1}{15} \times \frac{t_s}{F},$$

hence $$\frac{t_s}{F} = 18.4.$$

The speaker's formula, therefore, will be:

1.—For t_s smaller than $18.4\,F$: Use 1.4% of steel and fix the depth of the beam in accordance with Equation 7, that is, $m = h\, a\, d\, t_s$, substituting for h, 0.85, and for a, $\frac{1.4}{100}\, d$; hence,

$$d = \sqrt{\frac{84\, m^*}{t_s}} \quad \ldots\ldots\ldots\ldots \quad A$$

2.—For t_s larger than $18.4\,F$: Make depth of beam

$$d = \sqrt{m}\,(\alpha + \beta \times 10^{-6}\, t_s) \quad \ldots\ldots\ldots\ldots \quad B$$

* This, it will be observed, is simply another form of Thacher's formula.

Mr. Forchhammer.

The values of α and β, for various values of F are:

F.	α.	β.
1 900	0.040	0.282
2 100	0.038	0.244
2 300	0.036	0.218
2 500	0.034	0.200

The percentage of steel can then be fixed from Equation 7, using $h = 0.85$ and $m = 0.85\, a\, d\, t_s$, or

$$\frac{100\, a}{d} = \frac{100\, m}{0.85\, d^2\, t_s} = \frac{118\, m}{d^2 t_s}$$

$$= \frac{118}{t_s\, (\alpha + \beta \times 10^{-6} t_s)^2}$$

In these formulas:

m = the maximum moment per inch multiplied by the factor of safety required;

a = the area of the reinforcing rods per inch of beam;

d = the depth of the beam from the center line of the steel reinforcement to the ultimate fiber;

F = the ultimate compressive strength of the concrete;

t_s = the ultimate strength of the steel reinforcement (and for this the speaker proposes to use the elastic limit of the steel until it is plainly shown that a higher value is permissible).

Example 1.—A beam, 8 in. wide, is subject to a maximum bending moment of 100 000 in-lb. A factor of safety of 4 is required. The ultimate strength of the concrete is 2 200 lb. per sq. in., and the elastic limit of the steel is 30 000 lb. per sq. in.

$$m = 4 \times 100\,000 \div 8 = 50\,000.$$

As $\frac{t_s}{F} = \frac{30\,000}{2\,200} = 13.6$, and is less than 18.4, Equation A must be used.

Percentage of steel = 1.4.

$$d = \sqrt{\frac{84 \times 50\,000}{30\,000}} = 11.82 \text{ in., say 12 in.}$$

Example 2.—A beam, 6 in. wide, is subject to a maximum bending moment of 72 000 in-lb. A factor of safety of 5 is required. The ultimate strength of the concrete is 2 300 lb. per sq. in., and the elastic limit of the steel is 60 000 lb. per sq. in.

$$m = 5 \times 72\,000 \div 6 = 60\,000.$$

As $\frac{t_s}{F} = \frac{60\ 000}{2\ 300} = 26.1$, and is larger than 18.4, Equation B must be used. Mr. Forchhammer.

$$d = \sqrt{60\ 000}\,(0.036 + 0.218 \times 10^{-6} \times 60\ 000) = 12.0 \text{ in.}$$

$$\frac{100\,a}{d} = \frac{118 \times 60\ 000}{144 \times 60\ 000} = 0.82 \text{ per cent.}$$

If it is of importance to decrease the depth of the beam as much as possible, $\frac{t_s}{F}$ should be assumed as equal to 18.4. Then $t_s = 18.4 \times 2\ 300 = 42\ 300$. d can then be calculated either from Equation A or Equation B.

Using Equation A:

$$d = \sqrt{\frac{84 \times 60\ 000}{42\ 300}} = 10.9 \text{ in.; and } \frac{100\,a}{d} = 1.4 \text{ per cent.}$$

Using Equation B:

$$d = \sqrt{60\ 000}\,(0.036 + 0.218 \times 10^{-6} \times 42\ 300) = 11 \text{ in., and}$$

$$\frac{100\,a}{d} = \frac{118 \times 60\ 000}{121 \times 42\ 300} = 1.39 \text{ per cent.}$$

The results from these examples will be seen to check very closely with the results given in Table 5, and the speaker concludes that the equations proposed are sufficiently close approximations.

The speaker does not claim that the constants used for p, e, F, t_s, etc., will prove to be just the right ones. They may vary much, under different conditions, and only further experiments can fix their values.

To avoid any misunderstanding, the speaker wishes to state, finally, that the formulas proposed herein can only be applied where beams or floor slabs are exposed to actual bending. Wherever the arch action in the concrete is appreciable other formulas may be applied.

Arthur W. French, M. Am. Soc. C. E. (by letter).—This paper is full of interest for designers of reinforced concrete, and the author will receive the gratitude of all for his excellent work. Mr. French.

Considering the many theories and formulas for the design of reinforced concrete beams, the greatest difference of opinion seems to be in the matter of using safe stresses or ultimate stresses in the steel and concrete.

The author proposes that the design be based upon the ultimate strength of the beam. The writer prefers the use of safe working stresses in the materials.

It is common practice to design wooden and metal beams on the safe working stresses. Our formulas for flexure are theoretically correct for such beams when the stresses are within the elastic limits. Outside of these limits, although the same formulas are

Mr. French. used, it is recognized that they are purely empirical, and that they depend upon tests for the values of the constants.

It is not impossible to construct formulas, for the ultimate strength of beams of homogeneous materials, based upon the properties of the stress-strain diagrams of the materials, but it is generally considered impractical. If the stress-strain diagram of concrete were not quite such a smooth and attractive curve, often with a marked resemblance to a parabola, it is doubtful whether anyone would attempt a theoretical formula for the ultimate strength of a beam of this material reinforced with another material.

An examination of many diagrams for concrete will show considerable variation in the relation between strain and stress as the stress increases from zero to the breaking point. The lines often resemble the parabola with its axis at the breaking point, but, with the richer concretes, the lines become more nearly straight. In all these diagrams, if the attention be confined to the part of the line from zero up to any stress that would be used as a working stress, it will be seen to be sensibly straight. Moreover, the elastic deformation for stresses from 0 to 500 or 1 000 lb. has been well determined for many concretes, and probably with greater accuracy than for stresses near the breaking stresses.

With the assumption of a constant ratio of stress to strain within the limits of the working stresses (and the assumption is close to the truth and slightly on the safe side), the design may be made with as much certainty as in the case of wooden or steel beams. Let the ultimate strength of the beams be a matter of test and the results be expressed by an empirical formula.

Fig. 28 gives the formulas used for safe working stresses and a graphical solution showing the relations between the unit stresses in the steel and concrete for various percentages of reinforcement.

It is important to note that the exact position of the neutral axis is of little importance as far as the strength, calculated from the steel element, is concerned. The effective arm of the couple changes very little with widely differing assumptions as to the value of E_c, or of the shape of the diagram for deformation of the concrete, and but little with the percentage of reinforcement.

The depth of concrete under compression and the total compression in the concrete, however, is affected greatly by the position of the neutral axis and also by the assumed shape of the diagram for concrete. Fig. 29 gives the formulas for the assumption of a full parabola for the shape of the stress-strain diagram between the limits, zero and the working stress, and considers the coefficient of elasticity to be 3 000 000 and 2 000 000 at the origin of the curve.

Fig. 30 is also for the parabolic distribution of stress, but is based on coefficients of elasticity of 3 000 000 and 2 000 000 at the

Mr. French.

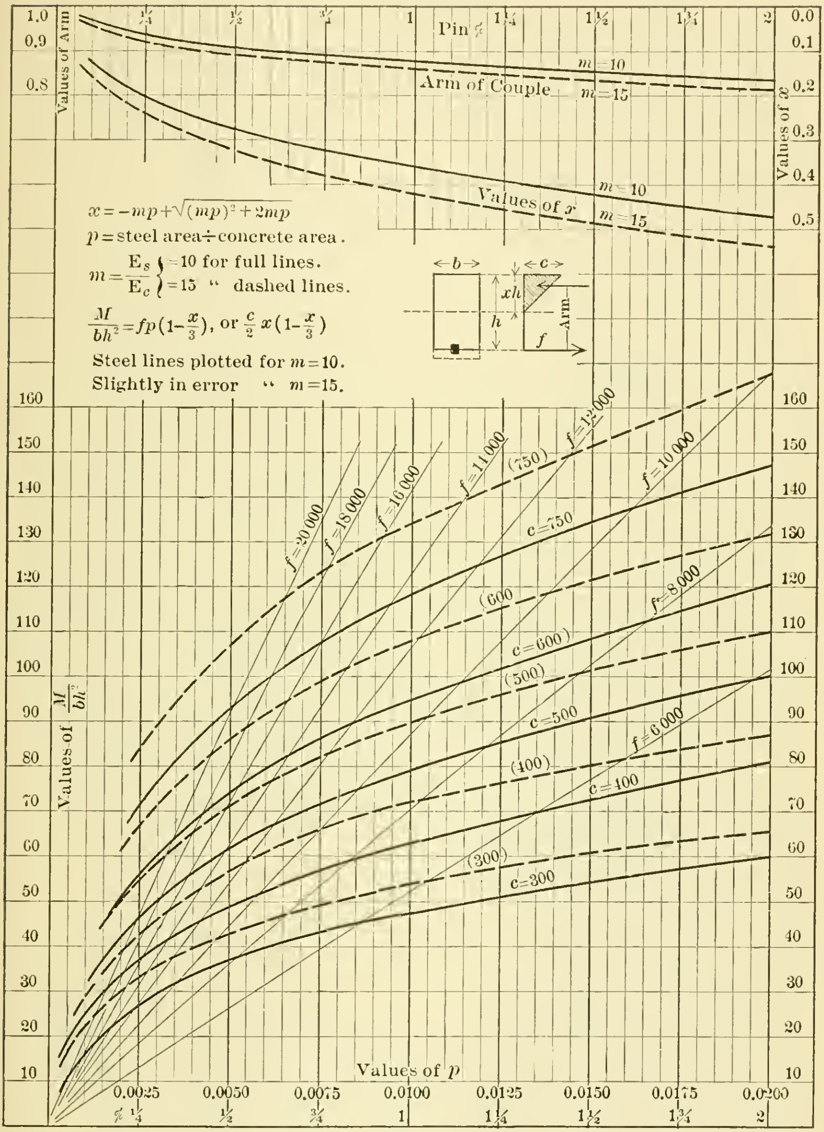

Fig. 28.

Mr. French.

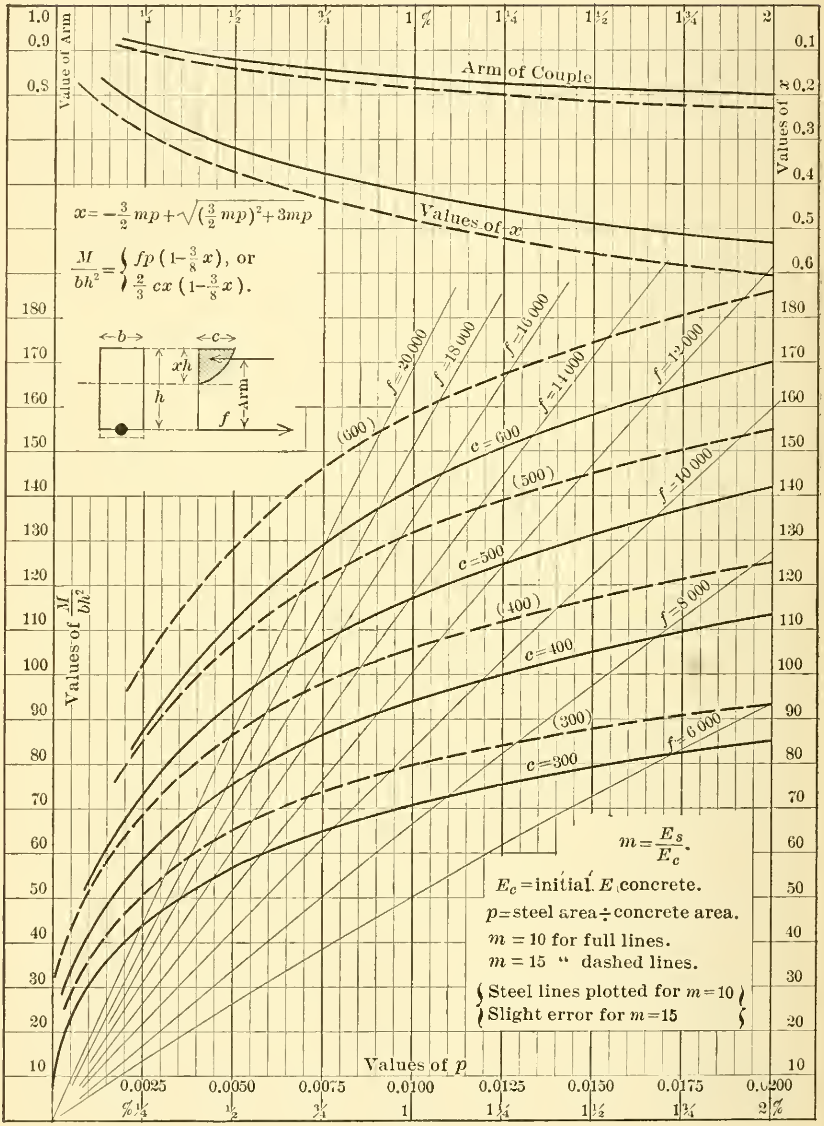

FIG. 29.

Mr. French.

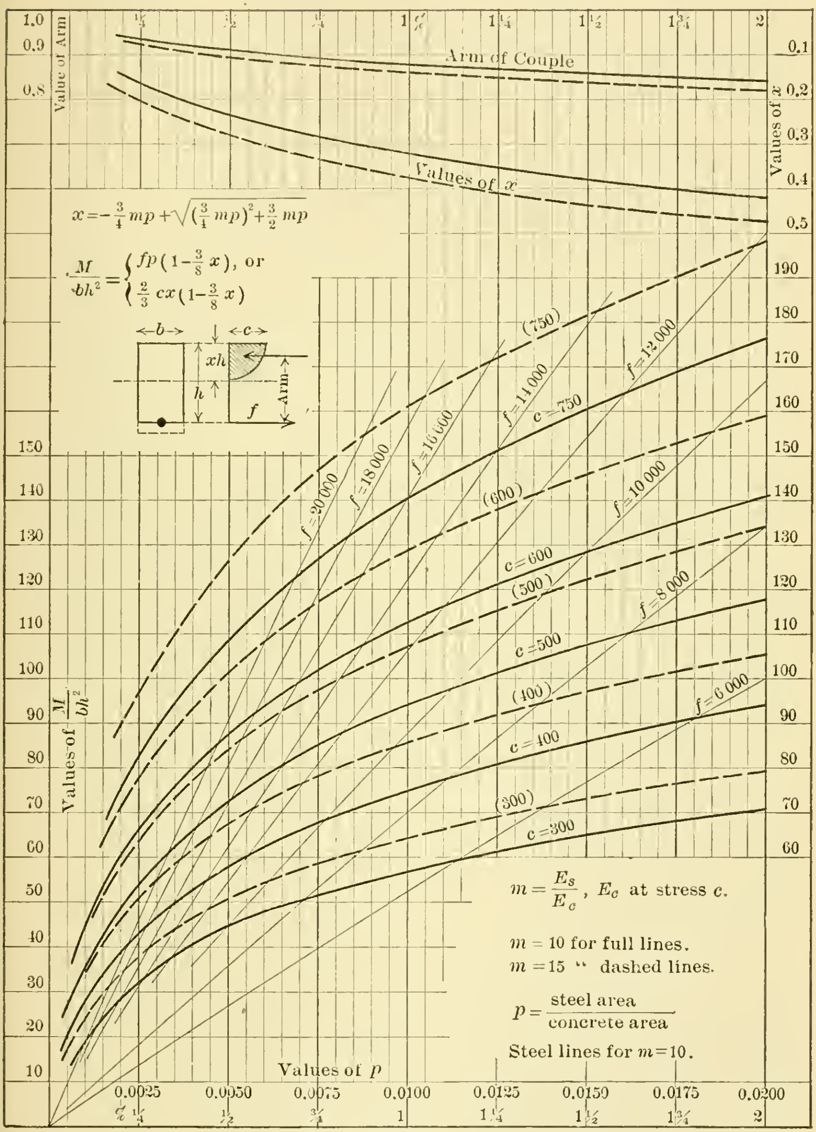

FIG. 30.

Mr. French. working stress. The assumption in the last case is equivalent to an initial coefficient of either 6 000 000 or 4 000 000.

The absurdity of assuming a full parabola for the stress distribution within a working limit of from 500 to 750 lb. has often been pointed out. Formulas based upon such an assumption may be made to give any desired results if the E is chosen properly, but they must be considered as purely empirical, and the writer would prefer an empirical formula of the correct form.

The writer cannot agree with the proposed formula of Mr. Wason for general application. The assumption of the neutral axis in the center of the beam is in close agreement with theory for the percentage of steel in the examples, and, used by its author or other expert, within certain limits, it gives good results. It is certainly the simplest formula proposed, and for about 1% of steel reinforcement, is of great value. Its use is equivalent to allowing 750 lb. stress upon the outer fibers of the concrete if the straight-line formula is used as a test. Probably many beams which are calculated by certain formulas to give 500 lb. stress have nearer 750 lb. if tested by the straight-line formula.

Practical experience with many structures of concrete tested up to 2 000 lb. per sq. in. in straight compression, as well as laboratory tests of such beams, seems to show that if designed by the straight-line formula, with from 600 to 750 lb. stress in the concrete, they will possess ample factors of safety. It may sound better to talk of 500 lb., rather than a higher stress, but the truth is the thing desired.

The use of the straight-line formula for working stresses will give lower percentages of steel allowed in the section, or if high percentages are used, the allowable stress in the steel is reduced automatically. As an example of the use of Fig. 28, if the stress in the concrete is to be limited to 600 lb., and $\frac{E_s}{E_c}$ is taken as 15, and the working stress permissible in the steel is 16 000 lb., the value of $\frac{M}{b\,h^2}$ will be 72, for $p = 0.005$. If p is 0.0065, the stresses will be 600 and 16 000 lb., and the value of $M \div b\,h^2$ will be 94. Where conditions demand, it may be wise to use higher percentages of steel, although this is not theoretically economical. Thus, if $p = 0.02$, the allowable stress in the concrete, of 600 lb., will determine the value of $M \div b\,h^2$ as 132, and the corresponding stress in the steel will be about 8 000 lb.

The natural limitations in the use of reinforced concrete beams and the desire for long spans or heavy load-carrying beams are constant temptations to over-steel the beams, and some formula which controls automatically the relation between the areas of steel and the stresses therein seems to be very desirable.

Shear reinforcement is shown to be needed in most beams, both Mr. French. by analysis and by tests, and it is well that increasing attention is being given to the subject. In view of the comparatively small amount of steel needed to insure safety from failure by shear, it is the writer's practice to be rather liberal with such reinforcement. If stirrups are to be used at all, the added expense of furnishing and placing a liberal supply is small.

Undoubtedly the most effective form of shear reinforcement is that of rods attached rigidly to the tension bars, sloping up toward the supports, and extending well to the top, or even extending into the slab. The one form of patented reinforcement which well fills the requirement of rigid attachment does not always easily supply the requisite number and length of shear rods where they are needed most. Other forms of rigidly attached rods seem to be clumsy and expensive. If the shear rods are to be loose stirrups, they should be set vertically. Ease in placing the concrete and in providing the desired number of rods of the proper length cause the writer to prefer, in practice, the loose vertical shear rods.

There would seem to be no reason why the width, b', of a slab assumed to act as a compression flange, should be limited to three times the width of the stem, b, if the proper reinforcement for shear at the section between the wings and stem be provided. Without reinforcement, the limit of $b' = 3\,b$ is well taken.

Irving P. Church, Assoc. Am. Soc. C. E. (by letter).—In making Mr. Church. a contribution to the discussion on Captain Sewell's valuable paper, the writer has in mind a brief, though fairly systematic, investigation of the relation between the cost and the dimensions, stresses, etc., of the ordinary concrete-steel beam or slab of rectangular section, with reinforcement on the tension side only. This treatment will be based on the assumption of the straight-line, stress-strain diagram for the concrete, and the tension of the concrete will be neglected (all loads and reactions being perpendicular to the beam and in the same plane). The notation adopted will be the same as that already used in the paper, but, for convenience, two additional symbols will be used, viz., r and e, representing the ratios, $\frac{T}{F}$ and $\frac{E_s}{E_c}$, respectively. The latter ratio will be taken as constant.

Since the distances, y_1 and y_2, are superfluous, as far as formulas for actual design are concerned, they will be eliminated at an early stage, while the compressive stress, F, in the "outer fiber" of the concrete need not be directly expressed in the final formulas, being replaceable when desired by the equivalent quotient, $\frac{T}{r}$. Again, T and F may have any values within reasonable limits, and the

Mr. Church. ratio, $\frac{T}{F}$, is not taken as equal to $\frac{t_s}{0.8 f_c}$, necessarily.

The author's Equations 1 and 2 still hold in the present connection, and from them is derived:

$$y_1 = \frac{e\,d}{r + e} \qquad \text{(1c)}$$

The centroid of the compressive stresses being at a distance of $\frac{2}{3} y_1$ from the neutral axis, the arm of the couple formed by the total stress, $a\,b\,T$, in the steel, and, $\frac{F\,b\,y_1}{2}$, in the concrete, is $d - \frac{y_1}{3}$, and hence

$$a\,b\,T\left(d - \frac{y_1}{3}\right) = M \qquad \text{(2c)}$$

The equality of the total stresses mentioned gives rise to the relation, $\frac{b\,F}{2}\,y_1 = a\,b\,T$, that is, $a\,b = \frac{b\,y_1}{2\,r}$, which, combined with Equation 1c, leads to

$$2\,r\,a\,(r + e) = d\,e \qquad \text{(3c)}$$

If y_1 is now eliminated from Equation 2*c*, by the aid of Equation 1*c*, and if for all subsequent work a value of unity be assumed for the width of the beam, so that the bending moment, M, becomes $m = \frac{w\,l^2}{8}$ (w being the uniform load per square inch of upper surface of the beam and l the span), there is obtained

$$3\,m\,(r + e) = (3\,r + 2\,e)\,T\,a\,d \qquad \text{(4c)}$$

Equations 3*c* and 4*c* are the only relations needed for design (aside from considerations of shearing stresses) for a beam of rectangular section with reinforcement on the tension side only. They constitute two independent equations, from which, if all the quantities concerned except two are given or assumed, these two may be determined, and hence should be regarded as constants.

If, however, all the quantities concerned in these two equations be given or assumed except three, these three are not determinate, but are variables; and by the aid of Equations 3*c* and 4*c*, a relation may be obtained between any two of the three variables; that is, any one of them may be expressed as a function of either of the other two.

For use in subsequent work the following relations, all derivable from Equations 3*c* and 4*c*, are here appended:

$$a = \frac{d\,e}{2\,r\,(r + e)} \qquad \text{(5c)}$$

$$a = \sqrt{\frac{3\,m\,e}{2\,r\,T\,(3\,r + 2\,e)}} \qquad \text{(6c)}$$

Mr. Church.

$$d = (r + e) \sqrt{\frac{6\ m\ r}{(3\ r + 2\ e)\ T\ e}} \ldots\ldots\ldots\ldots\ldots (7c)$$

The cost of a beam (of unit width), of fixed span and maximum bending moment, m, will now be expressed in terms of various variables, and the conditions of minimum cost investigated.

From the nature of the case, the only quantities which are not fixed at the outset, and hence would be available as variables, are four in number, viz., a, d, r, and T; so that four groups, of three in a group, may be selected, in turn, each group comprising three variables (m is treated as a constant).

As to the question of cost, consider only the items, (3) and (4) (mentioned on page 637* of the paper), assuming the cost of a cubic foot of steel to be equal to p times that of a cubic foot of concrete, and denoting by x a number proportional to the total cost of the two materials. Then, as in the paper,

$$x = p\ a + d \ldots\ldots\ldots\ldots\ldots\ldots (8c)$$

As a first group of variables take a, d, and T. Then, from Equations 5c and 8c, is found:

$$x = \left[\frac{p\ e}{2\ r\ (r + e)} + 1\right] d \ldots\ldots\ldots\ldots\ldots (9c)$$

which gives x as a function of the one variable, d.

Here it is seen that x is directly proportional to d, so that there is no mathematical minimum. Hence, for practical design, we have to take simply as small a value for d as may be consistent with a maximum safe value for T (one of the other two variables), while also ensuring a safe value for F, as implied in the assumed value of r.

For example, let $e = 16$, $r = 20$, $w = 8$ lb. per sq. in., $l = 100$ in. (so that $m = 10\,000$ in-lb.), and $T = 16\,000$ lb. per sq. in. With these values, then, from Equation 6c,

$$a = \sqrt{\frac{30\,000 \times 16}{40 \times 16\,000\ [60 + 32]}} = \sqrt{0.00815} = 0.0903 \text{ sq. in.},$$

and for d, from Equation 5c,

$$d = \frac{36 \times 2 \times 20 \times 0.0903}{16} = 8.12 \text{ in.}$$

This would make the percentage of steel $\frac{0.0903}{8.12} \cdot \frac{1}{100} = 1.10\%$ (referred to the concrete above the steel); and the value of F, implied in the assumption of $r = 20$, is $16\,000 \div 20 = 800$ lb. per sq. in.

Secondly, take r, a, and T, as variables. Equation 9c will serve in this case, also; that is,

$$x = \left[\frac{p\ e}{2\ r\ (r + e)} + 1\right] d \ldots\ldots\ldots\ldots\ldots (9c)$$

* *Proceedings*, Am. Soc. C. E., for December, 1905.

Mr. Church. giving x as a function of the one variable, r, d being a constant in this case. Evidently, x decreases with an increasing r. Here, again, there is no mathematical minimum for x (for any positive value of r) so that a practical use of the relation would be to take as large a value for r as would be consistent with a maximum safe value for T. For instance, with $T = 16\,000$ lb. per sq. in., $d = 10$ in., $e = 16$, and $m = 10\,000$ in-lb., it is found, from Equation 7c (solving by trial, since this proves to be a cubic equation), that the value of $r = 26.3$; which, in Equation 5c, gives a value of 0.0719 sq. in. of steel for a, implying 0.719% of steel (if the steel is compared with the concrete situated above it). With $r = 26.3$, the value of F would be 609 lb. per sq. in. With values of r smaller than 26.3, the cost would be greater, and both T and F would have smaller values than the above (in this case of constant d, etc., and variable r, a, and T); and neither material would be worked to its full (safe) strength.

Thirdly (and this is the most important and practical case), consider as the group of three variables, d, a, and r. If, in Equation 8c, there are substituted values for a and d from Equations 6c and 7c, there is obtained an expression for x as a function of the one variable, r, viz.,

$$x = \sqrt{\frac{3\,m}{2\,T\,e}} \left[\frac{p\,e + 2\,(r + e)\,r}{\sqrt{r\,(3\,r + 2\,e)}} \right] \ldots\ldots\ldots\ldots (10c)$$

To obtain a value of r for minimum cost, the differential coefficient of x with respect to r should be placed equal to zero, and the resulting equation solved for r; but this would be found to lead to an equation of such high degree that a simpler plan is to compute the value of x for each of a number of values of r and note the occurrence of a minimum x.

The writer has done this for seven values of r, ranging from 4 to 50, in an example in which there is given $m = 10\,000$ in-lb., $T = 16\,000$ lb. per sq. in., and $e = 16$; while p may be taken as 75 (that is, let 20 cents per lb. be the cost of the concrete, and \$15 that of the steel).

Since the variable part of the expression for x is the factor in the bracket, let the other (constant) factor be denoted by C. The following values for x have been computed:

For $r = 4$, $x = 102.6\ C$
" $r = 9$, $x = 71.6\ C$
" $r = 16$, $x = 62.2\ C$
" $r = 20$, $x = 61.6\ C$
" $r = 25$, $x = 62.9\ C$
" $r = 36$, $x = 69.6\ C$
" $r = 50$, $x = 81.8\ C$

Mr. Church.

It is seen that in this case x has a mathematical minimum, and that it is reached for a value of about 18 for r; and it is also noticeable that as r changes from 15 to 25 the variation in the cost is comparatively small.

Therefore, it may be said that in this example the cost is a minimum for $r = 18$.* Hence, with $T = 16\,000$ lb. per sq. in. the value of F would be 888 lb. per sq. in. If it were desired that the stress in the concrete should not exceed, say, 600 lb. per sq. in., this could be secured by adopting for r the value, 26.6, and the resulting cost would be but little in excess of that (the minimum) corresponding to $r = 18$.

As to the values of a and d, corresponding to $r = 18$ in this example, it is found, from Equation 7c, that $d = 7.54$ in.; and then, from Equation 3c, $a = 0.098$ sq. in. of steel; which is 1.31% of the part of the concrete above the steel.

As to the only remaining group of three variables, viz., d, T and r, it may be noted that the term, $p\,a$, in the expression for cost, $x = p\,a + d$, is constant (since a is assumed in this case), and that, consequently, the variable part of x is directly proportional to the height, d, and will be a practical minimum when d is made as small as possible consistent with a maximum safe value for T, one of the other variables. Hence, this case is virtually the same as one already treated. When the d has been computed for a proper T the corresponding values of r, and later that of F, are easily found.

Although the foregoing has been based on the simple assumptions of the straight-line, stress-strain relation, it is thought that the results would not have been very different if the parabolic form of curve had been used.

Mr. Leffler.

B. R. Leffler, Assoc. M. Am. Soc. C. E. (by letter).—The writer has had in mind for some time the publication of a paper on steel-concrete formulas, but Captain Sewell's paper has served a somewhat similar purpose. The writer wishes to add the following remarks in the way of a partial discussion, and addition; they apply to the subject in general.

Nearly all writers have adopted the ultimate-strength method of designing, but the writer believes this is a mistake. The evolution of rational designing shows a continual substitution of simple formulas of wide application for complicated ones of narrow application. For instance, there was a time when column formulas were entirely empirical. At present they are rational in form, with, perhaps, empirical constants. In fact, nearly the whole science of the mechanics of materials is derived from Hooke's Law—a simple law of wide application.

The common beam formula has been hotly disputed in the past, because it did not represent the actual conditions at the ultimate

* A later solution by the calculus gives 19.3 instead of 18.

Mr. Leffler. strength period. It would seem that reinforced concrete formulas are passing through a similar period of development; and that, finally, a simple formula, having a straight line in its stress-strain diagram, will supersede them all.

The writer's experience in reinforced concrete has been confined to the simple beam, and arches.

It is rather astonishing that, in the voluminous discussions of the past few years, no one has attempted to present a rational method of designing a reinforced concrete section for combined thrust and moment, on the supposition that steel takes tension only and concrete compression only. Edwin Thacher, M. Am. Soc. C. E., presented a formula, some years ago, in which concrete is supposed to take tension.*

Writers who adopt the ultimate-strength method of designing seem to be unaware of the impassable difficulties they create in a design for combined thrust and moment. They thus forever bar the rational design of reinforced concrete arches. As is well known, at any section of an arch there may be combined thrust and moment.

It may be well to state that all formulas and methods of designing arches are based on the assumption of a straight line in the stress-strain diagram.

The writer has before him a pamphlet—widely circulated throughout the Middle West—which purports to apply the ultimate-strength method to arches. The author of the pamphlet uses the common elastic theory for locating the pressure curve—a parabola in the case cited—but, in designing the individual sections for combined thrust and moment, he attempts to use the ultimate-strength method. These are some of the vagaries in the development of reinforced concrete design.

In designing the sections of an arch, the writer first used Thacher's formulas,† but abandoned these in favor of a method in which concrete takes no tension. He presents the following as an easy and rational method of designing a section for combined thrust and moment.

Let Fig. 31 represent two short blocks, having steel on one side, as shown, and subject to an eccentric compressive load. This is a typical case in arch design. Then, granting a straight line in the stress-strain diagram, no tension will occur in the steel as long as the load acts within the middle third of the concrete section. The small strip of concrete on one side of the steel is neglected.

When the load is at the far edge of the middle third, the compression on the far edge of the concrete is twice the average, and tension in the steel is about to take place.

A movement of the load into the far third produces further com-

* *Engineering News*, September 21st, 1899.

† *Transactions*, Am. Soc. C. E., Vol. LV, p. 188.

pression in the concrete, and causes tension in the steel. It is evident that this extra pressure, and tension in the steel, is directly proportional to the distance of the point of application of the load beyond the far edge of the middle third. Mr. Leffler.

For simple beams, the writer uses the straight-line formulas, as given in "Concrete, Plain and Reinforced," by Taylor and Thompson.

The following is the writer's simple method of designing for combined thrust and moment:

The moment, for extra pressure at the far edge of the concrete and for tension in the steel, is equal to the distance of the point of application of the load beyond the far edge of the middle third,

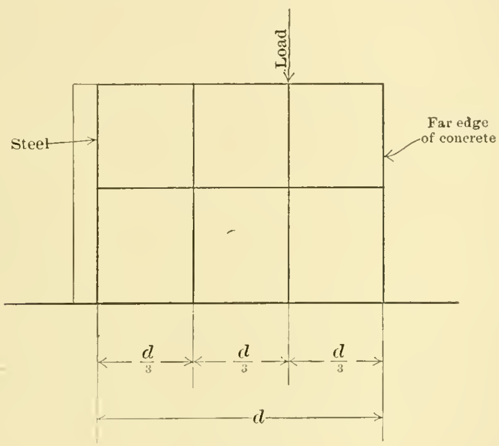

FIG. 31.

multiplied by the load. Use this moment as if it occurred in a simple beam, determining the extra pressure on the concrete, and tension in the steel. Add the extra pressure to twice the average pressure for the total pressure on the concrete at the far edge.

It is well to remember that the middle-third theory is based on a straight line in the stress-strain diagram, and cannot be used in the ultimate-strength method of designing.

The writer would like to see an advocate of the ultimate-strength method of designing attempt to show when steel takes tension in the above case!

The assumption of a straight line in the stress-strain diagram leads to simple formulas of wide application, and hence is bound to prevail. Of course, in this assumption, working stresses should be used, and not ultimate stresses.

Mr. Leffler. It seems to the writer that the present field of engineering literature on reinforced concrete is being turned into a vast mathematical gymnasium. An engineer is not primarily a mathematician, but only incidentally so.

Mr. Hill. GEORGE HILL, M. AM. SOC. C. E. (by letter).—Although reinforced concrete has been in use now for some few years, and the literature on the subject is becoming voluminous, a lack of agreement is still found in regard to facts. Relatively few tests have been made, and there are imperfections in the observations, tending to minimize the value of these tests, the majority of which have been for commercial rather than engineering purposes.

In glancing over the literature on the subject, the reader will notice that as a writer gains experience in the actual use of reinforced concrete his views regarding it change. He is more wary in using indiscriminate tests, is apt to make more for himself, and is less certain that he can solve all problems theoretically. This would indicate the desirability of less work with pencil and paper and far more work with a testing machine, before the application of the differential calculus to the results.

In certain tests made by the writer* he was struck by the similarity, in reasonably good slabs, of what might be called the load curve. All these tests were made with a portable hydraulic testing machine having a recording apparatus. Pains were taken to secure a reasonably uniform application of the load, and to record the deflection constantly, so that for each test there was a graphical representation which spoke directly to the eye and conveyed lessons which would otherwise have been lost unless considerable labor were expended in plotting the results. All the load curves indicated an elastic limit point for the combination, which also appeared in the deflection curve. In the writer's opinion, tests which stop short of the destruction of the object tested are of no value, and these constitute the bulk of the public tests during the past five years. Tests which do not indicate clearly the behavior of the piece under each increment of load, and those made for some especial purpose, without photographs, are of but little value, as the observations are open to dispute; therefore one can understand the author's desire for further tests.

Another serious difficulty is the lack of uniformity in the concrete. In the writer's opinion, therefore, the two most fruitful subjects for present investigation are the production of a concrete by commercial methods having fairly uniform characteristics which can be predetermined, and the testing to destruction of reinforced concrete members, under conditions which will secure graphic

* *Transactions*, Am. Soc. C. E., Vol. XXXIX, p. 617.

records of the load, its rate of application, the deformation at each instant, and the appearance of the piece tested at certain critical periods. Mr. Hill.

The author, on page 636,* states as follows:

"Therefore it seems justifiable to assume that a formula such as Equation 6 can be made just as accurate as any of the forms of Equations 1 to 5, especially if its constants be determined from tests designed with that end in view."

With this, the writer is in full accord. To a considerable extent, it supports the view he expressed in his paper† presented in April, 1898. While engineers may feel that it is too simple a solution for men of their mathematical attainments, yet, as they are charged with the duty of getting full value for their employers' time as well as their own, they should realize that the present state of the art admits of nothing better.

The writer doubts very much the value of the theoretical discussion of maximum economy by the methods of the calculus. Not only does the cost of the aggregate vary in different localities, but it varies constantly in the same locality. Owing to the beneficent rule of the labor unions, one may in one locality pay for labor at $1.75 per day, to-day, and six months later be required to pay $4 per day for the same work. During one month it may be found that one form of deformed steel bar can be had cheapest, and the next month some other form. It may be permitted on one job to use a certain style of form, or centering, which is relatively inexpensive and can be used repeatedly, but the next job may be so different that none of the former centering is available. Therefore, it follows that whatever may be the most advantageous for one job may be very uneconomical for the following one. The writer believes that no formula can be devised which will cover these conditions, and be used by a busy man.

In the ninety-four tests published in the writer's paper, previously referred to, he observed in no case any necessity for web reinforcement, and, in properly designed commercial structures, he doubts the necessity or desirability of such reinforcement. In relatively deep and slender T-beams it may be necessary, but it is questionable whether or not it is commercially desirable to design such beams.

The writer is heartily in accord with the following statements by the author: on page 626,* in regard to the impression of the extreme accuracy of the complicated formula, which is not at all justified; on page 630,* in regard to the desirability of taking conservative values for the maximum allowable stresses; on page 634,* in regard

* *Proceedings*, Am. Soc. C. E , for December, 1905.

† *Transactions*, Am. Soc. C. E., Vol. XXXIX, p. 617.

Mr. Hill. to the experimental determination of the maximum allowable percentage of reinforcement; on page 636,* heretofore referred to; on page 640,* that the best that can be done is to use as large a percentage of steel as the concrete will stand; on page 651,* that a mechanical bond is to be preferred to simple adhesion; on page 656,* that it is of importance that the floor slab and the part of the beam or girder below it should be bonded together; and paragraph 6 on page 659.*

In regard to the statement, on page 654,* concerning the dehydration of cement, the facts may be as stated by the author; the conclusion that this damaged material must be removed does not follow, for, if the tensile strength of the cement is neglected in the computations, this material is only used as a protection for the steel, and if it performs that function it might as well be left. It will perform that function if a wire netting or other similar substance be used for the sole purpose of retaining the cement around the reinforcement. It is to be earnestly hoped that this paper will attain its main object, as explained in the last paragraph on page 659.*

Engineers who design reinforced concrete should come to an agreement in regard to the most desirable method of making tests; in regard to practicable formulas which can be applied in design; in regard to formulas applicable to floor slabs with numerous supports; in regard to the shrinkage of slabs and walls in setting; in regard to a provision for either the elimination or localization of cracks; and in regard to the commercial limitations which, in general, should apply in the design and construction of reinforced concrete buildings.

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* *Proceedings*, Am. Soc. C. E., for December, 1905.

Vol. XXXII. APRIL, 1906. No. 4.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NEW FACTS ABOUT EYE-BARS.
Discussion.*

By Messrs. Henry B. Seaman, Mansfield Merriman, Albert J. Himes, A. W. Carpenter and John Thomson.

Henry B. Seaman, M. Am. Soc. C. E.—The matter of chief interest in connection with these full-sized tests is that, on 50-ft. bars, they offer a confirmation of the specimen tests made by Bauschinger years ago. Mr. Cooper's tests on eye-bars show a permanent set at a strain of about 12 000 lb. per sq. in., while his specimen tests indicated an elastic limit of about 35 000 lb. If his eye-bars had been still longer, it is possible that a permanent set would have been observed at even a lower strain. This, to the speaker's mind, is a very satisfactory confirmation of the deduction of Bauschinger that, after a strain is once applied, the elongation is never entirely eliminated, although it may gradually decrease if allowed time for rest. Mr. Seaman.

These tests, the speaker believes, confirm the statement made by him in a paper† upon the Launhardt formula, that the experiments

* This discussion (of the paper by Theodore Cooper, M. Am. Soc. C. E., printed in *Proceedings* for January, 1906), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Communications on this subject received prior to May 26th, 1906, will be published subsequently.

† *Transactions*, Am. Soc. C. E., Vol. XLI, pp. 141 and 146.

Mr. Seaman. of Wöhler entirely destroy the theory of the perfect elasticity of metals as formerly accepted, and require that the term be abandoned and a new definition sought. Since that date the term, "yield point," has gradually replaced the old term, "elastic limit."

Mr. Cooper's experiments are valuable in demonstrating the necessity of considering the results of refined testing in large structures.

Mr. Merriman. MANSFIELD MERRIMAN, M. AM. SOC. C. E.—The full-sized drawings exhibited by the author show the distortions in the eye-bar heads more clearly than the speaker has heretofore seen. From these lines it is possible to study the actual distribution of the stresses throughout the metal, and probably a more precise knowledge might be obtained than that which we now possess. The lines, ruled on the head before making the test, were parallel and normal to the length of the bar forming 1-in. squares, and the distortions of these squares indicate the nature and the relative intensities of the stresses. Where a square is seen to be distorted into a rectangle, one side being shorter and the other longer than 1 in., it is known that there existed compressive and tensile stresses at right angles to each other. Where a square is distorted into a rhombus, it is known that shearing stresses also prevailed. The speaker regards these drawings as of much interest and value, and hopes that the author may be able to publish one or more of them for the benefit of the engineering profession.

Referring now to the general question brought forward by the author, it seems to be proved by the tests that the elastic limit of the eye-bar, as a whole, is reached before that of the bar proper. It is not difficult to see that this is entirely due to the high compressive stress in the eye-bar head at the back of the pin, this being due to the small bearing surface between the pin and the head. The usual rule for determining the bearing compressive stress, by dividing the total tensile load by the diametral area of the pin hole, is, of course, a rough approximation, and it is certain that, with the usual clearances, the actual stress between the pin and the eye-bar head is very much greater than given by this rule. As a consequence, the compressive elastic limit of the metal in the head is exceeded before the tensile elastic limit of the metal in the bar proper is reached. Fortunately, the shape and size of the eye-bar heads have been so proportioned by experiment that rupture almost always occurs in the bar, and hence the author's conclusions throw no distrust upon the eye-bar system of bridges, as far as safety is concerned. His investigation, however, is of value and importance in computing the camber of long spans, and also for cases where the deflections of the ends of projecting trusses require to be computed.

In order to decrease the compressive stress in the metal back of the pin, it has been suggested to increase the thickness of the head of the eye-bar, and also to use a harder steel for the head. While the head can be made thicker, it is doubtful if it would be expedient to do so with such large eye-bars as those used in the Quebec Bridge. The use of harder steel for the head does not seem practicable unless the bar itself is also of the same grade of steel; in this case the elastic limits of both head and bar would be higher, the allowable unit stresses would also be taken higher, and, hence, the same phenomena as before would occur. Mr. Merriman.

A third method that has been suggested is to cut the eye-bar hole of oval shape, the shorter diameter of the oval being a little larger than the diameter of the pin, while the longer diameter, which is parallel to the axis of the bar, is sufficiently large to give ample clearance. The curvature of the oval at the back of the pin should be greater than that of the pin, so that the pin, when first brought into bearing, does not quite touch the back surface of the hole, but bears along the head at two places on each side. The curve to be used should be such that, for a certain tensile stress in the bar, say, 15 000 lb. per sq. in., the radial compressive stresses between the pin and the head would be closely equal over an arc of 120°; if this can be attained, the intensity of the radial compressive stress will be less than 18 000 lb. per sq. in. The theoretic determination of this curve is not an easy matter, for the pin, also, is deformed as the stress increases, but a few experiments would undoubtedly result in producing an oval hole for which the distortions of the head would be very much less than those shown in the author's drawings. While the cutting of such holes would add somewhat to the cost of the eye-bars, it may be noted that the difficulty of inserting a pin through many bars in erection would be much diminished, since the oval holes would furnish ample longitudinal clearance.

ALBERT J. HIMES, M. AM. SOC. C. E. (by letter).—That an eye-bar is known to be permanently elongated in the pin-hole when the strain in the body is not more than 12 000 lb. per sq. in. is a startling fact, and should have been discovered before. In now bringing the matter before the Society, Mr. Cooper has added one more important service to the generous list which he has already given to the profession. Mr. Himes.

Although it is not found in practice that bridges are developing a deflection such as would be caused by elongation of the pin-hole, and, in taking down numerous old bridges which have been subjected to loads far beyond those contemplated in their design, no deformation of the pin-hole has been noticed, these facts merely demonstrate, as in the case where one of a pair of eye-bars carries the whole load, that the assumptions of loading aud factor of safety

Mr. Himes. are so liberal that defects, like this lack of strength in the pin-holes, have not produced any noticeable effect in working structures.

Such defects, however, are elements of weakness which greatly reduce the supposed factor of safety and render of small value the liberal sections brought into use by some of the impact formulas.

The author's discovery will also do much good indirectly by checking a tendency toward over-confidence in the perfection of the art of bridge building. That there is still something to discover is very evident, and the need of greater caution is plainly indicated.

A theoretical discussion of the effects on the pin-hole of tension in the bar gives results so much in accordance with those described by the author that it will be presented for consideration.

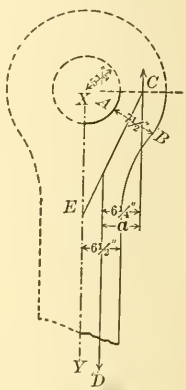

Fig. 7.

If an eye-bar be imagined to be divided longitudinally on the center line, and each half of the bar to carry its proportion of the tension independently of the other half, and then, if the head of the bar be cut through the center of the pin-hole at right angles to the axis of the bar, the result is a free body, shown in Fig. 7. This free body is acted upon by only two forces: tension in the body of the bar and a parallel tension in the head; but the two forces are separated by a distance, *a*, thus forming a couple. The moment of this couple must be resisted by a section of the head, *A*, *B*, and, in a specific case, the computation of the outer fiber stresses in this section will show that the usual working stress in the body of the bar produces stresses in the section which exceed the elastic limit.

Assume a bar 1 in. thick.

Assume a unit stress of 12 000 lb.

Tension at $D = 6\frac{1}{2} \times 12\ 000 = 75\ 000$ lb.

" " $C = 75\ 000$ lb.

Arm of couple $= 6\frac{1}{4}$ in.

Moment of couple $= 469\ 000$ in-lb.

Section $A\ B = 7\frac{1}{2} \times 1$ in. $= 7\frac{1}{2}$ sq. in.

Moment of inertia of $A\ B = 35.2$.

$$\text{Outer fiber stress at } A = \frac{M\ e}{I} = \frac{469\ 000 \times 3.75}{35.2} = 49\ 960 \text{ lb.,}$$

which exceeds the elastic limit.

This condition agrees precisely with those reported by the author. He discovered compression at *B* and elongation at *A*, and a permanent deformation when the unit tension at *D* was 12 000 lb.

With the change of shape of the pin-hole, there must come a re-distribution of stress in the section, *A B*, so that the tension at *A* will decrease and the compression at *B* will be changed to tension. If the direction of the tension at *C* be changed so as to pass through the head and intersect the axis of the bar at *E*, there is no longer a couple, and the tendency to deformation which it produced in the section, *A B*, has ceased; or, it may be said that a second couple has been formed by the lateral pressure against the pin and its corresponding resistance in the section, *X Y*, this couple acting in a direction opposed to the first couple, and therefore relieving the bending stress in the section, *A B*. This condition agrees well with the fact that after a slight stretch at *A*, the pin-hole is not generally ruptured, although the bar breaks in the body under a tension four or five times as great as that which caused the first deformation in the pin-hole. Mr. Himes.

Mr. Cooper discovered that a slight variation in the pin-hole clearance produced no appreciable effect in the deformation of the bar, and this fact also agrees with the theory, since a maximum variation of, say, $\frac{1}{16}$ in. is very small, compared with the arm of the couple.

In attempting to meet the requirement that bars tested to destruction shall break in the body rather than in the head, the manufacturers, apparently, have rested content with their success, and have paid little attention to the character of the deformation.

If the section, *A B*, could be given a moment of resistance great enough to keep the tension at *A* well below the elastic limit, a condition which exists in the riveted flat bar, Fig. 4,* the defect would be corrected. Another remedy would be to alter the shape of the pin-hole and head to conform approximately to that due to final distortion. The latter method would not be perfect, but it would greatly lessen the defect. The bar would still have to stretch enough to come to a bearing on the sides of the pin, after which it might be fairly assumed that the bending moment has been eliminated. The manufacturers would find some difficulty in making the elongated holes, but the tests appear to indicate that an improvement is needed.

The author's conclusion, that bars of high tensile strength are to be preferred because they exhibit less deformation in the tests, would seem to be unsound, because such bars would be strained to the yield point in the section, *A B*, as well as bars of softer material, and, if steel must be deformed, it is well known that the softer grades are safer.

While eye-bars are under consideration, the writer desires to say something in reference to annealing. There seems to be a prevalent idea that the full-sized test is satisfactory if the bar does not break

* *Proceedings*, Am. Soc. C. E., for January, 1906, p. 24.

Mr. Himes. in the head. That result is assumed to prove the success of the bridge shop; and previous specimen tests have shown the character of the mill product.

Fractures partially crystalline are very common, and brittleness sometimes appears. These defects, in all probability, are due to heat treatment, and, as the bars have been annealed, they cannot be charged to the rolling mill. The full-sized test should be a test of annealing as well as a test of the workmanship on the heads, and there can be no true test of annealing unless a bar is broken from every charge of the annealing furnace.

The annealing of eye-bars has long been subject to the personal skill and supposed infallibility of men who, though faithful and skilful beyond the average, have, nevertheless, a poor conception of the scientific properties of steel. The importance, in annealing, of a uniform and rapid heating to a temperature, not too high, and of uniform and fairly rapid cooling, is not generally understood. Ridsdale has shown the effects of too high a temperature and of chilling,* and bars that bore all the evidence of such treatment have been seen by the writer. It would seem that the substitution of a pyrometer for the time-honored color test would afford a more certain control of the temperature and be another step in the march of progress.

Mr. Carpenter. A. W. Carpenter, Assoc. M. Am. Soc. C. E. (by letter).—The author states that the failure of the usual assumptions, as shown by his investigation, is not of much importance in ordinary bridges on account of low unit stresses. It would seem to the writer that the stresses in ordinary bridges are frequently, if not generally, high enough to come within the range of those which are shown to be serious. With the increase of loads and with the impact, in the case of railroad bridges, the nominal stresses for which the structures are designed are greatly increased, and the details should be such as to take care of any increase in the stresses as well as in the main sections.

In view of the results obtained by the author, the present generally-adopted design of eye-bar heads is defective, even for ordinary structures, especially as the tendency is toward smaller heads. The author's experiments were conducted upon bars with heads larger in proportion than are now furnished in ordinary practice. The excess percentage through the eyes of the bars tested is shown to vary from 39 to 69%, with one exception, in which the excess was 31 per cent. The largest bridge concern in the country has, for a standard, a head with 30% excess of section through the eye, and guarantees the development of the full strength of the bars with such heads. It would appear from the author's tests that the

* *Engineering News*, Vol. 46, pp. 238 and 276.

stretch of the pin-holes in such heads, due to stresses within working limits, would obtain in greater degree than with the larger heads. It would seem, therefore, that the manufacturers should change their standards to produce larger heads, even at the expense of some metal and room for clearance. This would seem to be a primary step in the right direction. As pointed out by the author, however, something more is necessary. He shows that reducing the pin clearances and increasing the size of the pins does not affect the results. Mr. Carpenter.

He calls attention to the superiority of harder steel, and the tests appear to confirm this superiority. The writer believes that the steel used for eye-bars and other annealed members should be of a harder grade than that used for unannealed material, so that the finished work may be more nearly of the same strength throughout. With the added advantage of stiffening the pin-holes, this would surely seem to be the proper selection of material. This, of course, is in line with the author's suggestion, his other recommendation being to stretch the eyes longitudinally before final boring. The latter may be practicable, but seems to be a little doubtful.

The writer would offer the following suggestion: that the heads of the bars be made thicker than the bodies, a method which was extensively used at one time. This would seem to be the most efficient method of decreasing the maximum pressure of the pins on the pin-holes. This pressure, owing to the necessary clearance of the pin, however infinitesimal, must be a variable pressure, having a maximum at the back of the pin-hole in the line of stress. This maximum pressure is reduced directly in proportion as the head is thickened. The section in the head could thus be very rapidly increased without increasing the diameter, and the manufacture of such heads would seem to present no difficulties. The disadvantages would be in the increased space occupied in packing, the increased length, and, probably, in the increased strength required for the pins. A minor advantage in the thickening of the heads would be the greater separation of the bodies of the bars, as these sometimes lie too close for painting. It is possible that a very slight thickening would give the desired result, but this is a matter which it would only seem possible to prove by experiment.

An old handbook of the Phœnix Iron Company gives a table of dimensions of thickened eye-bar heads, which shows the range of thickness varying from $\frac{1}{4}$ to $\frac{5}{8}$ in. for bars up to 6 in. in width. The excess section obtained varies from 43 to 87 per cent. The material, of course, was iron, and the writer understands that the heads were formed partially by piling and welding, and partially by upsetting.

The writer's suggestions, summarized, would be, to make some bars with heads of the usual circular shape, and with a section through the pin-hole 50% in excess of the body of the bar, using

Mr. Carpenter. medium steel running to the highest limit of tensile strength (70 000 lb. ultimate strength), with heads thickened, say, 25% over the body of the bar, and test these for the stretch of the pin-holes on the lines followed by the author. Some change in the ordinary design and method of manufacture should be made to remedy the defect pointed out.

Mr. Thomson. JOHN THOMSON, M. AM. SOC. C. E. (by letter).—The following observations, while not derived from a line of application similar to that described by Mr. Cooper, may yet have some interest, and indicate a line of further experimentation which, if carried out properly, may cast additional light upon the subject.

As to the statement:

"We have assumed that a set of bars carefully bored to an exact length would all pull to an equal strain, as long as the elastic limit measured on the body of the bar was not exceeded."

The writer can say that in his experience with short connecting rods, used for heavy duty on printing and embossing machinery, it has been known, for a considerable time, that the design of the eyes and the relative proportion existing between the bearing surfaces thereof and the pins upon which they act, are factors of the first importance.

Thus, if the eye-bars described in the paper are regarded as connecting rods to be used in tension on a machine, then, in the writer's opinion, the reason they failed in the manner set forth would be due to the fact that the bearing surfaces, as between the bores of the eyes and the pin, have not sufficient area.

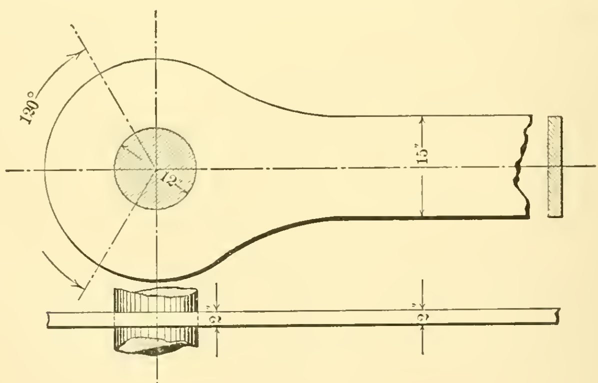

FIG. 8.

Fig. 8 is a view of a 15-in. bar, 2 in. thick, with a 12-in. pin. The effective arc of contact on such an eye and pin will not exceed 120°; if loosely fitted, as stated, it will hardly exceed, say, 90°, which

is the arc of contact in primary intimate contact. But, assuming the maximum, or 120°, the effective area in contact to resist the pull of the bar will be approximately 25 sq. in. The area of the rod, 15 $\times$ 2 = 30 sq. in., which, when subjected to a stress of 24 000 lb. per sq. in., gives a total test load of 720 000 lb.; and this, divided by the area of the bearing, gives an average pressure of 28 800 lb. per sq. in. of that surface, or 4 800 lb. per sq. in. in excess of the tensile stress per square inch in the main body of the eye-bar. The elastic limit of the steel is not given in the paper, but it may be assumed as being not far from the pressure developed within the eye upon the pin at the test-strain quoted. Be this as it may, it is a fact that, with the relative proportions adopted, the intensity of pressure, even upon the most favorable assumption of conditions, is greatest where it should be the least. Mr. Thomson.

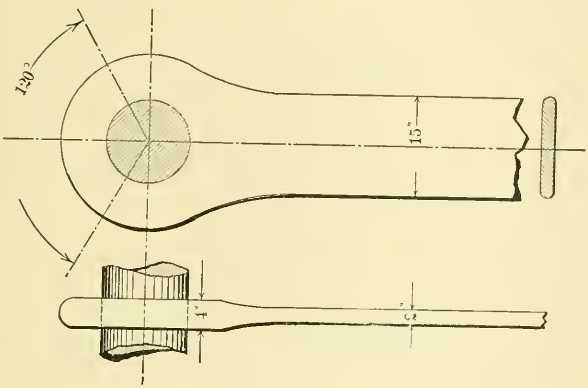

Fig. 9.

The remedy, assuming that the adopted cross-sectional area of the main body of the bar is essential, is to increase the area of the surfaces in contact between the inner surface of the eye and the bearing pin, and, in the writer's opinion, for such a purpose, the extent of this bearing should be approximately twice that of the cross-sectional area of the main body of the bar. This can be obtained in two ways: First, by considerably increasing the diameter of the pin and eye; or, second, by increasing the thickness of the eye. The latter method is regarded as more preferable. Such a construction is shown in Fig. 9, the outside diameter of the eye being decreased and its thickness doubled. The mass of metal is approximately the same in either instance. In this way the effective bearing surface is doubled, that is, it is 50 sq. in.; and, under a loading similar to that cited, the pressure per square inch would be 14 400 lb., or 9 600 lb. per sq. in. less than the tensile stress in the main body of the

Mr. Thomson. bar and 14 400 lb. per sq. in. less than that in the bar of Fig. 8. In the writer's judgment, this feature is the key to the problem. It may not be quite so "handy" for rolling-mills to slab out bars of the form indicated in Fig. 9, but this would probably be "all to the good," as there appears to be no reason why bars of the dimensions given in the paper should not have their eyes formed by forging, or hydraulic pressure, in forming-dies, as has been done most successfully, in thousands of instances, in the writer's own experience. In this way, too, there is another advantage in that the forged bore of the eye can be swaged, relatively cold, thus condensing and hardening the metal where it bears upon the pin.

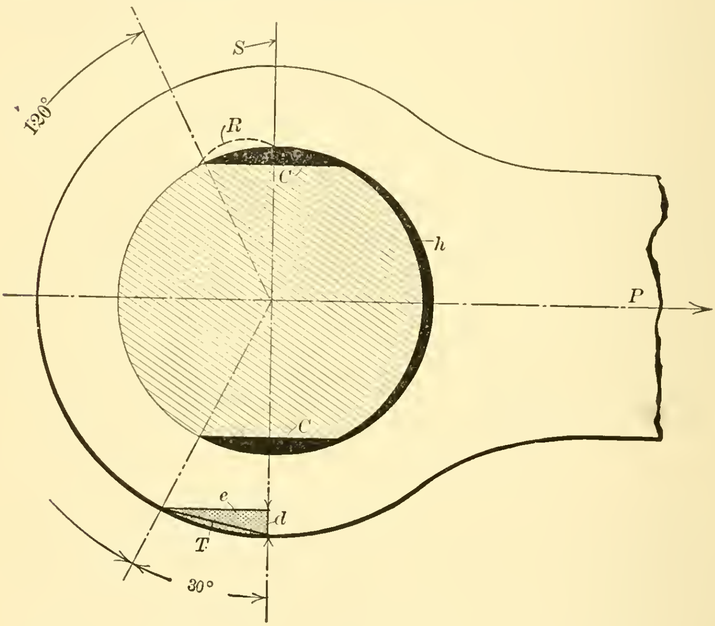

FIG. 10.

There is another point relative to this matter, which, however, is presented with some hesitancy. It is illustrated by Fig. 10. Here, the query is: Would it, or would it not, be advantageous to flatten the pin at the top and bottom, line S, at right angles to the line of strain, P? As to whether or not this detail is new, the writer does not pretend to say, although he does not know of its having been adopted outside of his own practice. For several years past, this detail has been used especially in bearings, from 12 to 15 in. in diameter and with 2 to 3-in. face, in embossing presses subjected to exceedingly heavy duty. Prior to making this modification, a num-

ber of these rods had failed, having fractured through the forward quadrants of the eyes, where, it may be observed, practically all such fractures take place, at least in the service now being considered. Since making the change in the bearings, that is, planing the flats, C, at the top and bottom of the journal or pin, not a single eye has parted, although the duty demanded has since been considerably increased. What is the reason? This the writer does not pretend to answer definitely as the result of actual demonstration, that is, demonstration undertaken for the express purpose of proof, but his theory as to the cause may be stated as follows: Mr. Thomson.

When the eye of the rod is subjected to such a stress that it is stretched away from the free side of the bearing, as h, or, what amounts to the same thing, if the forward bearing surfaces wear or yield under compression, this produces motion at the top and bottom of the journal or bearing-pin; and, as a considerable portion of the bearing, in these locations (20°, 30°, 40°), is but slightly effective in resisting strain directly, applied as at P, these segments act as highly effective wedges, to augment the direct or normal strain, and operate to burst the forward quadrants of the eye. This so-called "wedge" is denoted, on the lower edge of Fig. 10, on an arc of 30°, in which the bursting effect would be as the relation of the versed sine d to the sine e; or, say, about five times that of the primary strain, friction being disregarded. Obviously, the same result would be obtained by slightly clearing the eye, as at R, or by a less flattening of the pin, as denoted by the line T. It may be mentioned that these clearances, in a revolving bearing, afford excellent means for lubrication, and permit a preliminary fit, upon the circular arcs, of the journal or pin, considerably closer than would otherwise be permissible.

Whether the foregoing theoretical explanation stands or falls, the proof of the effectiveness of the principle in practical use, in the application cited, is complete; and the writer would have no hesitation in utilizing it under any analogous condition. In other words, paraphrasing a portion of Mr. Cooper's opening text, "hold fast that which is good," whether or not one finds theories to fit the case. This, however, is not intended to mean that it is not somewhat better to have a close working combination between theory and practice, which is intended to apply especially to Figs. 8 and 9 and the description relative thereto.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THE PANAMA CANAL.

Discussion.*

By Messrs. George B. Francis and Theodore Paschke.

Mr. Francis. George B. Francis, M. Am. Soc. C. E.—Among the arguments in favor of a canal of least first cost, viz., a lock canal, the speaker has not seen any of the following kind, and therefore introduces it as a matter which may be of interest.

No man possesses such a prophetic vision that he can forecast (beyond a very limited number of years) what the future will bring forth. The future tendency, however, can be judged by reasoning from the experience of the past.

In all construction pertaining to transportation by roads, railroads, and canals, a limited number of years has brought about, from various causes, large and important changes. Sometimes routes and construction works have been entirely abandoned. In other cases, the capacity of the work has been many times enlarged. Again, the amount of funds at first available has seemed ridiculously small after traffic had developed to such an extent that adequate construction could be carried out.

At the end of fifty years, the generation then existing has often ridiculed the inadequate conceptions of the original constructors. Knowing from experience the incapacity of mankind to foresee the requirements of the future, beyond a reasonable period, is it wise to

* This discussion (of the paper by A. G. Menocal, M. Am. Soc. C. E., printed in *Proceedings* for February, 1906), is printed in *Proceedings* in order that the views expressed may be brought before all members for discussion.

Communications on this subject received prior to May 26th, 1906, will be published subsequently.

look too far ahead, or to expend much more money than will produce a reasonable waterway, within a reasonable time, and at reasonable first cost? Mr. Francis.

Is it good judgment to be concerned about the way in which this canal will be altered from a lock to a sea-level canal, in the future, or to spend any money to attain that object?

In the absence of knowledge as to the amount of traffic or the possible alteration and increase in the size and draft of vessels, is it wise to prepare for extremes? On one hand, the lock canal may be adequate for many years; on the other hand, it may be entirely inadequate in a short time.

When the time comes for enlargement, engineers will have new ideas, and, instead of enlarging or lowering the canal in its original location, it may be that a new location, perhaps relatively close to the present one, perhaps as far away as Nicaragua, will be thought more desirable; perhaps two canals, at just such a great distance apart, will be considered better than a larger one in one locality. Perhaps two canals adjoining each other, each taking traffic in one direction only, will be preferred. Even though locks are made so that the canal may be changed to sea-level, the traffic may be so great, when the time to make the change comes, that it will be altogether impracticable, on account of the serious interruption to traffic, or the necessity for temporary abandonment of traffic altogether.

A great number of examples could be cited in support of such conditions, not only in transportation lines, but in public works like fortifications, river improvements, and many minor kinds of construction.

Fifty or one hundred years from now, the wealth and energy of the United States will be beyond the possible conception of any person now living, and the expenditure of $100 000 000, which is a large sum to-day, will seem to be relatively small to the statesman or engineer of the future.

Looking at the whole problem of the Panama Canal from this point of view, is it not good judgment to build now a good, substantial lock canal for substantially the needs within view, or, in the event of a decision to build either a sea-level or a lock canal, to expend no appreciable amount of money thereon in preparation for changes or enlargements which may be remote, and may be made in a manner entirely different from that now conceived?

THEODORE PASCHKE, M. AM. SOC. C. E.—The special feature of Mr. Menocal's paper is an argument in favor of a lock canal, culminating in the somewhat novel proposition of the combined viaduct-dam at Gamboa. Mr. Paschke.

However interesting this feature may be to the lock-canal partisans, it will be well to defer its full consideration until a necessity

Mr. Paschke. therefor has arisen, and until it has been fully decided that a lock canal shall be built. This opportunity is taken to express the hope that that time may never come.

To the speaker, this paper seems to be a very timely invitation to the engineers of the country to express their opinions on the all-important question which at present is before the United States Government for decision; and it behooves this Society to speak freely on the question as to the type of canal which should be adopted.

It is with this in view that the speaker ventures a few remarks on this subject.

Mr. Menocal says:

"It is evident that a lock canal is the most economical type, both in cost and time of construction, and that the sea-level proposition is born * * * of sentiment," etc.

The question of comparative economy between the two types is fully covered, and may best be left to be answered by the testimony given before the Senate Committee by J. F. Wallace, Past-President, Am. Soc. C. E., the late Chief Engineer of the Canal.

As to the matter of the sea-level proposition being born of sentiment, why, the speaker, for one, on the sea-level side, admits this at once, and without hesitation. All achievements in the march of civilization, of improvement, are born of sentiment. Was not the voyage across the Atlantic of that most intrepid of all navigators, Christopher Columbus, which resulted in the discovery of this continent, born of sentiment? And how about the Declaration of Independence? And still later, the preservation of the Union? Were these not born of sentiment? Here is a whole chain of grand, noble sentiments, one after the other, growing out of the primary sentiment entertained by the Genoese ancient mariner, culminating finally in the reasonable expectation of the realization of his dream of more than four hundred years ago. Why, then, should sentiment detract from the sea-level proposition?

The speaker is inclined to remind the author that we are at Panama, not at Nicaragua. If he were at the latter place, he would be justified in sweeping aside the sea-level proposition as born of sentimentality. But, at Panama the sentiment of the sea-level proposition becomes tangible and within the range of practicable execution. Yea, more, the sentiment becomes conviction that the sea-level proposition is the best and the only proposition worthy of consideration.

All will admit that the underlying sentiment of an ideal type for a ship canal across the Isthmus is a free and unobstructed passage of ships, and all, even the most fervent enthusiasts of the lock proposition, will admit that a lock in a ship canal is an obstacle, a hindrance, to such free and unobstructed passage, notwithstanding

the labored paradoxical arguments advanced in the minority report Mr. Paschke. of the consulting engineers. Now, reducing the question of superiority of type to a simple arithmetical proposition, there is the lock-canal proposition with four or more obstacles in the way of a free and unobstructed passage of ships, as against the sea-level proposition with only one lock, and that one open for one-third of the time. There is the underlying sentiment of the sea-level propo-School.

There is no desire on the speaker's part to make light of the engineering difficulties in the way of a realization of the proposition for a sea-level canal, but he has faith in the resourcefulness, in the genius of American engineering talent to solve all problems in connection with the proposition completely and successfully, when confronted with the mandate of the nation.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

JAMES MacNAUGHTON, M. Am. Soc. C. E.*

DIED DECEMBER 29TH, 1905.

James MacNaughton was born on January 6th, 1851, in Albany, New York, and died on December 29th, 1905, in New York City. He was the son of Dr. James MacNaughton, Dean of the Faculty of the Albany Medical College.

His early education was received at the Albany Boys' Academy, Albany, New York, from which institution he was graduated in 1867. After a year's study, he entered the sophomore class of Yale College, and was graduated with honors from the classical department in July, 1871. While there he paid particular attention to the study of mathematics, and had conferred upon him the second senior mathematical prize of his class.

After graduation he accompanied Professor Marsh as a member of his party on a geological exploration in Kansas, Colorado and Wyoming. On his return to Albany from this expedition he studied chemistry and other scientific branches at the Albany Medical College. In 1873 he entered the Rensselaer Polytechnic Institute, and took a special course in technical engineering subjects for a period of two years.

In 1875, immediately after leaving the Rensselaer Polytechnic Institute, he was appointed Rodman on Mr. C. L. McAlpine's party on surveys for the new aqueduct for New York City. He was engaged in the field, and also on the office work in connection with this survey, and remained with the party until the completion of the maps, estimates, etc., in the fall of the next year.

In April, 1876, he was appointed Rodman in the Department of Public Works, New York City, in connection with the construction of the new storage reservoir near Brewster, New York. Soon after his appointment to this position, he was made Leveler on the same work, and was thus engaged for about a year and a half.

In the autumn of 1877, he was promoted to be an Assistant in charge of the surveys for the location of a new storage reservoir, east of Brewster Station, but in the latter part of December, 1877, he resigned and returned to Albany, where he took charge of superintending and getting out the plans for the Hotel Kenmore, in Albany. He had charge of this work as Superintending Engineer

* Memoir prepared by James C. McGuire, Assoc. M. Am. Soc. C. E.

of construction, and in January, 1879, as soon as the work was completed, he went abroad, and for four months was engaged in study in the École des Ponts et Chaussées, at Paris.

He returned to the United States in October, 1879, and shortly thereafter was engaged as Assistant Engineer on the West Shore Railroad for about two years.

In 1885 he was appointed an engineer on an expedition sent out by the Canadian Government on H. M. S. *Alert,* which made explorations and surveys on the Hudson Bay Coast.

Mr. MacNaughton was a member of the Association for the Preservation of the Adirondacks, and took much interest in the development of the forests and the cutting of timber from large tracts of land. In 1903 he took a course at the Yale Forestry School.

He was elected a Member of the American Institute of Mining Engineers in 1890. The late President Blanco of Venezuela decorated him for services in that country, in connection with certain engineering enterprises. He was a Member of the New York Board of Trade and Transportation of New York City.

Just prior to his death, he did much to develop the manufacture of Ferro-Titanium on a commercial basis, having been President of the Ferro-Titanium Company which built a plant at Niagara Falls and successfully manufactured Ferro-Titanium for the market. He received much recognition for his work in this line, both in the United States and abroad, and it is entirely due to his efforts that the use of this alloy has been made possible from a commercial standpoint. He was also President of the MacIntyre Iron Company at the time of his death.

Mr. MacNaughton was a member of the Arts Club, the University Club, and the Down Town Association, of New York City; the Tahawas Club, of Essex County, New York, the Fort Orange Club, and the Albany Country Club, of Albany, New York.

It is to be particularly noted that both in his business connections, and in the societies and clubs of which he was a member, he was always conspicuous for his dignified bearing and courteous treatment of all who came in contact with him, and especially those under him. He never married.

While his loss to his friends is great, his loss to the scientific world is even greater, for he was engaged in the development of properties and industries of which his knowledge was so complete, and for which he had done so much that it will be impossible to fill his place. His death is mourned by all who knew him or who came in contact with him, either as a friend or as a citizen.

Mr. MacNaughton was elected a Member of the American Society of Civil Engineers on May 5th, 1880.

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

May, 1906.

PROCEEDINGS - VOL. XXXII—No. 5

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

HERMAN W. SPOONER

Published at the House of the Society, 220 West Fifty-seventh Street, New York, the Fourth Wednesday of each Month, except June and July.

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL. XXXII. No. 5.

MAY, 1906.

Edited by the Secretary, under the direction of the Committee on Publications.

CONTENTS.

NEW YORK 1906.

American Society of Civil Engineers.

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Vice-Presidents.

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E. KUICHLING.	BERNARD R. GREEN.

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Treasurer, JOSEPH M. KNAP.

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Term expires January, 1907:	*Term expires January, 1908:*	*Term expires January, 1909.*
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The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays. Fourth of July, Thanksgiving Day and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER: - - - 533 Columbus.
CABLE ADDRESS: - "Ceas, New York."

Vol. XXXII. MAY, 1906. No. 5.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

SOCIETY AFFAIRS.

CONTENTS:

MINUTES OF MEETINGS.

OF THE SOCIETY.

May 2d, 1906.—The meeting was called to order at 8.35 P. M.; President Frederic P. Stearns in the chair; Chas. Warren Hunt, Secretary; and present, also, 130 members and 24 guests.

The minutes of the meetings of April 4th and 18th, 1906, were approved as printed in the *Proceedings* for April, 1906.

Stacy B. Opdyke, Jr., M. Am. Soc. C. E., moved that the following resolution be referred to the Board of Direction (Art. VI, Sec. 12, of the Constitution):

"*Resolved,* That a Special Committee be appointed to collect such information as may be obtainable on the present and prospective status of the adoption of the metric system in the United States, and to collate and systematize such information and promulgate it to the Society in form of reports from time to time."

The motion, being duly seconded, was adopted by a vote of more than twenty-five Corporate Members.

A paper, by William W. Harts, M. Am. Soc. C. E., entitled "The Control of Hydraulic Mining in California by the Federal Government," illustrated with lantern slides, was read by the Assistant Secretary. The Secretary presented correspondence on the paper from Messrs. H. H. Wadsworth, F. Riffle and J. D. Galloway, and the subject was discussed orally by Richard Lamb, M. Am. Soc. C. E.

Ballots for membership were canvassed, and the following candidates elected:

As Members.

Paul Goodwin Brown, New York City.
James Francis Cullen, Havre de Grace, Md.
John Henry Dockweiler, San Francisco, Cal.
Chauncey Eldridge, Boston, Mass.
Norman MacPherson Hench, Pittsburg, Pa.
Horace Longuet Higgins, Manila, Philippine Islands.
Albert Harrison Hogeland, St. Paul, Minn.
Daniel Webster McMorris, Corregidor Island, Philippine Islands.
Charles Fillmore Mebus, Philadelphia, Pa.
Eugene Francis Musson, Norwich, N. Y.
Walter Howard Sawyer, Lewiston, Me.
Charles Herbert Vaughan, New Brighton, Pa.
Luther Wagoner, San Francisco, Cal.

As Associate Members.

Warren Martin Archibald, Nashville, Tenn.
Wilford Willis DeBerard, Harrisburg, Pa.
Louis Harvey Ehrbar, New York City.
Albert Preston Greensfelder, St. Louis, Mo.
Edward Creswell Heald, Washington, D. C.
Oskar Augustus Johannsen, Ithaca, N. Y.
Masayoshi Kabashima, Kansas City, Mo.
George Smith Green Lewis, New York City.
Washington Irving Lex, Pencoyd, Pa.
Herbert Prescott Linnell, Cristobal, Canal Zone, Panama.
Armour Cantrell Polk, New Orleans, La.
Frank Forrest Sinks, Chicago, Ill.
James Bean Wilson, Chicago, Ill.
Joseph Johnson Yates, Elizabeth, N. J.

As Associate.

Robert Hileman Lee, Kingston, R. I.

The Secretary announced:

The transfer of the following candidates, by the Board of Direction, on May 1st, 1906:

FROM ASSOCIATE MEMBER TO MEMBER.

OSCAR FRANCIS BELLOWS, Albany, N. Y.
HOWARD NICHOLAS EAVENSON, Gary, W. Va.
FOSTER HAVEN HILLIARD, Memphis, Tenn.
WILLIAM STONE JOHNSON, Boston, Mass.
FREDERICK THOMAS LLEWELLYN, New York City.
SANFORD ELEAZER THOMPSON, Newton Highlands, Mass.
EDWARD DE VOE TOMPKINS, New York City.

The election of the following candidates by the Board of Direction:

AS JUNIORS.

On April 3d, 1906:

GOUVERNEUR CADWALADER, Philadelphia, Pa.
GEORGE NOBLE COPLEY. Galveston, Tex.
CHESTER CENTENNIAL FISHER, Rupert, Idaho.

On May 1st, 1906:

FREDERICK BAYARD BARSHELL, New York City.
MAX JOHN BARTELL, San Francisco, Cal.
ROY BULLEN, Rupert, Idaho.
LOUIS CHEVALIER, Detroit, Mich.
ELIHU CUNYNGHAM CHURCH, New York City.
FRANK GILLELEN, Los Angeles, Cal.
EDWARD HAROLD HOPSON, Pawling, N. Y.
JOHN MARVIN PETERS, Connellsville, Pa.
FREDERICK HORACE TIBBETTS, San Francisco, Cal.
ADONIRAM JUDSON WARLOW, Wilkes-Barre, Pa.

The Secretary read a letter he had lately received from Otto von Geldern, M. Am. Soc. C. E., Secretary of the Technical Society of the Pacific Coast, stating that the great need of the engineers of San Francisco was drawing instruments, paper, and office supplies. The dealers having been burned out, such articles could not be purchased, and even if they could be secured there was no money to pay for them.

On motion, duly seconded, the Secretary was authorized to expend a sum, not to exceed $1 000, in the purchase of supplies for the prompt relief of the engineers of San Francisco.

The Secretary announced that a circular will soon be issued in regard to the arrangements for the Annual Convention at the Hotel Frontenac, Thousand Islands, on June 26th to 29th, 1906, and that the following topics for informal discussion have been chosen for that meeting:

1.—What is the best Preparatory Education for the Civil Engineering Profession?

2.—Is Technical Training the Best Education for Executive Work?

3.—The Protection of the Intellectual Property of Civil Engineers.

4.—The Advance in Sewage Disposal.

5.—What are the Best Means for the Prevention of Conflagrations in Large Cities?

6.—The Filtration of Water.

Adjourned.

May 16th, 1906.—The meeting was called to order at 8.35 P. M.; H. F. Dunham, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, 156 members and 64 guests.

A paper, by George B. Francis and W. F. Dennis, Members, Am. Soc. C. E., entitled "The Scranton Tunnel of the Lackawanna and Wyoming Valley Railroad," was presented by Mr. Francis and illustrated with lantern slides.

The paper was discussed by Messrs. F. Lavis, V. H. Hewes and George B. Francis.

The Secretary announced that, in accordance with the action of the Society at its last meeting, he had purchased and forwarded to the Technical Society of the Pacific Coast $500 worth of drafting instruments and office supplies for the engineers who had lost everything in the disaster at San Francisco. He also read letters from members in San Francisco expressing their satisfaction with the action of the Board of Direction in offering to replace any Society publications lost by any member in the fire at very small cost.

The Secretary announced the death of CHARLES LOUIS SPIER, elected Associate May 3d, 1905; died May 7th, 1906.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

May 1st, 1906.—President Stearns in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bissell, Bowman, Ellis, Gowen, Knap, Kuichling, Lewis, Noble, Schneider, Sherrerd, and Smith.

The Secretary was authorized to send as complete a set of *Transactions* as possible to the Technical Society of the Pacific Coast to replace the files lost by fire.

The Secretary was authorized to furnish to any member who has lost his volumes of *Transactions*, through the San Francisco disaster, duplicates at a discount of 75% from the list price.

The Secretary was directed to communicate with the President of the San Francisco Association of Members of this Society expressing sympathy with our members in San Francisco, and asking in what manner the Society can best aid engineers in that city during the present crisis.

Applications were considered, and other routine business transacted.

Seven Associate Members were transferred to the grade of Member, and ten candidates for Junior were elected.*

Adjourned.

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* See page 169.

ANNOUNCEMENTS.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

MEETINGS.

Wednesday, June 6th, 1906.—8.30 P. M.—A regular business meeting will be held. Ballots for membership will be canvassed, and a paper, entitled "Disposal of Municipal Refuse, and Rubbish Incineration," by H. de B. Parsons, M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in *Proceedings* for April, 1906.

Wednesday, September 5th, 1906.—8.30 P. M.—A regular business meeting will be held. Ballots for membership will be canvassed, and a paper, entitled "Concerning the Investigation of Overloaded Bridges," by Wilbur J. Watson, M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in *Proceedings* for April, 1906.

Wednesday, September 19th, 1906.—8.30 P. M.—At this meeting a paper, entitled "Street Traffic in New York City, 1885 and 1904," by Clifford Richardson, Assoc. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings*.

ANNUAL CONVENTION.

The Thirty-eighth Annual Convention of the Society will be held at The Frontenac, Thousand Islands, Frontenac, N. Y., on June 26th to 29th, 1906.

The general arrangements for the Convention are in the hands of the following Committee:

CHARLES S. GOWEN,
JOHN W. ELLIS, MORRIS R. SHERRERD,
J. WALDO SMITH, CHAS. WARREN HUNT.

A Circular in regard to the Convention has already been issued.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all meetings:

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Society of Engineers, 17 Victoria Street, Westminster, S. W., England.

American Institute of Mining Engineers, 99 John Street, New York City.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Civil Engineers' Club of Cleveland, 1200 Scofield Building, Cleveland, Ohio.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Philadelphia, 1122 Girard Street, Philadelphia, Pa.

Engineers' Society of Western Pennsylvania, 410 Penn Avenue, Pittsburg, Pa.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Engineers' Club of Central Pennsylvania, Corner, Second and Walnut Streets, Harrisburg, Pa.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Teknisk Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Société des Ingénieurs Civils de France, 19 Rue Blanche, Paris, France.

Svenska Teknologförenigen, Brunkebergstorg 18, Stockholm, Sweden.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Sachsischer Ingenieur- und Architekten- Verein, Dresden, Germany.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Pacific Northwest Society of Engineers, 617–618 Pioneer Building, Seattle, Wash.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Memphis Engineering Society, Memphis, Tenn.

Oesterreichischer Ingenieur= und Architekten=Verein, Eschenbachgasse 9, Vienna, Austria.

The Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Cleveland Institute of Engineers, Middlesbrough, England.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

SEARCHES IN THE LIBRARY.

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling, compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

Copies of all lists of references are filed, so that in many cases it is only necessary to make a typewritten copy, which reduces the cost of searches to a minimum.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

ACCESSIONS TO THE LIBRARY.

From April 9th to May 7th, 1906.

DONATIONS.*

THE CONQUEST OF ARID AMERICA.

By William E. Smythe. New and Revised Edition. Cloth, 8 x 5 in., illus., 26 + 360 pp. New York, The Macmillan Company, 1905. $1.50 net.

The preface states that the author has endeavored to show the peculiar environment of the arid region and the influence which it will exert on the civilization of our Western land: the lessons to be learned from the more notable of the early pioneer settlements in Colorado, Utah and California; the natural advantages and present development of the great States and Territories between the Missouri River and the Pacific Ocean; the beginning, progress, and triumph of the National irrigation movement and the work of the corps of young men organized into the United States Reclamation Service.

The book is intended to be of value to the investor, the tourist, the economist, the legislator, the reader of history and travel, and those interested in American resources and institutions generally—but. most of all, the author hopes that it will be of some practical use to men and women who are looking for homes in the West. There is an index of ten pages.

VENTILATION OF BUILDINGS.

By William G. Snow and Thomas Nolan. Cloth, 6 x 4 in., 83 pp. New York, D. Van Nostrand Company, 1906. 50 cents.

The authors have tried to condense the statement of the general principles of ventilation and of their application to different kinds of buildings. The details of the mechanics of ventilation have been purposely omitted, as they are discussed in another volume of this series. It is hoped that the book may prove useful, not only as a popular presentation of the subject for the general public, but also as a suggestive outline in architectural, engineering and other schools, in connection with or introductory to the whole subject of ventilation. The Contents are divided into three parts: I.—General Principles of Vent lation; II.—Different Systems of Ventilation; III.—Ventilation of Different K nds of Buildings. There is no index.

YARDS AND TERMINALS AND THEIR OPERATION.

By J. A. Droege. Cloth, 9 x 6 in., illus., 285 pp. New York, The Railroad Gazette, 1906. $2.50.

The author states that the relative importance of the freight terminals of a line of railroad is not usually understood, and the attention they deserve is not always given them. As improved terminals result in a more prompt and cheaper handling of freight, and tend to a more general utilization of freight lines, many railroads are adding to or remodeling existing yards. The newly constructed or "model" yard is mainly interesting, therefore, in that it affords a guide for the revision, extension or remodeling of the old yards. The first chapter of the book is devoted to the relative importance of terminals and main lines in point of cost of operation and linear feet of rail used, with the terminal problem of Greater New York given as an example. In the second chapter, the author recommends the adoption of the terms and definitions relating to terminals and yards compiled by a committee of the American Railway Engineering and Maintenance of Way Association. In the chapters on the designing of terminals, extracts from the report of the Committee on Yards and Terminals of the same Association are taken as a basis of discussion. Several chapters of the book are devoted to the operation and operating forces of terminals and yards, which may be read with interest by railroad men generally. The instructions as to the rapid handling of fast freight and the construction and location of freight houses and piers are given in two chapters. The book is illustrated with line cuts and half tones. The Contents are: Relative Importance of Terminals; Terms and Definitions; General Principles of Design; Designing Terminals; Track Details; Ash Tracks; Coaling Plants; Icing Plants; Switching Methods; Pole Switching; Summit Switching; Gravity Switching; Records; Management and Discipline; The Yardmaster; Loading Cars; Making Up Trains; Fast Freight; Freight Houses; Freight Piers; and Coal Piers. There is an index of six and one-half pages.

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* Unless otherwise specified, books in this list have been donated by the publisher.

Gifts have also been received from the following:

Am. Electrochemical Soc. 1 bound vol.
Am. Locomotive Co. 4 pam.
Am. Soc. of Mech. Engrs. 1 vol.
Am. Steel & Wire Co. 4 pam.
Baltimore, Chesapeake & Atlantic Ry. Co. 2 pam.
Brit. Fire Prevention Committee. 2 pam.
Brooklyn Engrs. Club. 1 bound vol.
Brooklyn Public Library. 1 vol.
Byllesby, H. M., & Co. 1 pam.
Canada-Geol. Survey. 3 pam.
Case School of Applied Sci. 1 vol.
Cay, M. D. 1 drawing.
Cincinnati Southern Ry. Co. 1 bound vol.
Cleveland, Cincinnati, Chicago & St. Louis Ry. Co. 1 pam.
Colo.-Agri. Exper. Station. 5 pam.
Columbus, Ohio-Highway Dept. 1 pam.
Fall River, Mass.-Watuppa Water Board. 1 pam.
Holyoke, Mass.-City Engr. 1 pam.
Illinois Univ.-Agri. Exper. Station. 2 pam.
Incorporated Assoc. of Municipal and County Engrs. 1 bound vol.
India-Public Works Dept. 3 bound vol., 2 vol.
International Assoc. of Municipal Electricians. 1 bound vol.
Jewell Export Filter Co. 1 pam.
Lake Superior Min. Inst. 1 vol.
Luster, W. H. 1 pam.
Mass.-Met. Park Comm. 1 bound vol.
Merchants' Assoc. of San Francisco. 1 pam.
Met. West Side Elev. Ry. Co. 1 pam.
Mexican Inter. R. R. Co. 1 pam.
Missouri-R. R. and Warehouse Commrs. 1 bound vol.
Municipal Engrs. of the City of New York. 1 pam.
New Bedford, Mass.-Water Board. 1 pam.
New Jersey-State Board of Health. 1 bound vol.
New York City-Board of Health. 1 pam.
New York Central & Hudson River R. R. Co. 1 pam.
New York Chamber of Commerce. 1 bound vol.
New York City Record. 1 bound vol.
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BY PURCHASE.

Elektrotechnik in Einzeldarstellungen, Vols. 6-7. Herausgegeben von G. Benischke. Braunschweig, Friedrich Vieweg und Sohn, 1905.

Reports. Permanent International Association of Navigation Congresses. 21 pam. Brussels Printing Office of Public Works (Co. Ltd.), 1905.

The Nile in 1904. By William Willcocks. London, E. & F. N. Spon, Limited; New York, Spon & Chamberlain, 1904.

National Engineering and Trade Lectures, Vol. II. British Progress in Pumps and Pumping Engines. By Philip R. Björling. London, Archibald Constable & Co., Ltd., 1905.

National Engineering and Trade Lectures, Vol. III. British Progress in Gas Works' Plant and Machinery. By C. E. Brackenbury. London, Archibald Constable & Co., Ltd., 1905.

The Prevention of Senility and a Sanitary Outlook. By Sir James Crichton-Browne. London, Macmillan & Co., Limited; New York, The Macmillan Company, 1905.

A Text-Book on Gas, Oil, and Air Engines. By Bryan Donkin. Fourth Edition, Revised and Enlarged. London, Charles Griffin and Company, Limited, 1905.

Gasworks Accounts and Management. By George Helps. London, The *Gas World* Offices, 1905.

Hydraulique Agricole et Urbaine. Par G. Bechmann. Paris, Ch. Béranger, 1905.

Alternating Currents, Their Theory, Generation and Transformation. By Alfred Hay. New York, D. Van Nostrand Company, 1906.

Metallurgical Calculations. By Joseph W. Richards. New York, McGraw Publishing Company, 1906.

SUMMARY OF ACCESSIONS.

From April 9th to May 7th, 1906.

Donations (including 5 duplicates and one number completing volume of periodical)	164
By purchase	32
Total	196

MEMBERSHIP.

ADDITIONS.

MEMBERS.

		Date of Membership.
BELLOWS, OSCAR FRANCIS. Asst. Engr., N. Y. State Barge Canal, Barge Canal Office, DeGraff Bldg., Albany, N. Y.	Assoc. M. M.	Dec. 4, 1901 May 1, 1906
BROWN, PAUL GOODWIN. 461 Lexington Ave., New York City		May 2, 1906
EAVENSON, HOWARD NICHOLAS. Chf. Engr., United States Coal & Coke Co., Gary, McDowell Co., W. Va.	Assoc. M. M.	Mar. 6, 1901 May 1, 1906
FROSELL, CARL GUSTAF. Engr., Am. Bridge Co., Pencoyd, Pa.		April 4, 1906
HARING, ALEXANDER. 2305 Loring Pl., University Heights, New York City		April 4, 1906
HUGGINS, WILLIAM. Care, Wm. Krug & Son, 42 Rua S. Bento, Sao Paulo, Brazil		Feb. 7, 1906
JOHNSON, WILLIAM STONE. Asst. Engr., State Board of Health, Room 140, State House, Boston, Mass.	Assoc. M. M.	June 6, 1894 May 1, 1906
LUIGGI, LUIGI. 81 Via Vardegna, Rome, Italy		Feb. 7, 1906
MENDEN, WILLIAM STEPHEN. 752 Westminster Rd. (Flatbush), Brooklyn, N. Y.		April 4, 1906
OLIVER, EMERY. Div. Engr., West. Pac. Ry. Co., Oroville, Cal.		Mar. 7, 1906
ROSS, ELMER WAYLAND. Asst. Engr., Bridge Dept., City Engr.'s Office, Providence, R. I.	Jun. Assoc. M. M.	Mar. 5, 1890 June 1, 1892 April 3, 1906
SELTZER, HARRY KENT. Res. Engr., Waddell & Hedrick, I. & G. N. R. R., Bridge Reconstruction, Austin, Tex.	Jun. Assoc. M. M.	Feb. 4, 1896 Jan. 2, 1901 April 3, 1906
SHAW, ENOS LARKIN. 2667 N. Ashland Ave., Chicago, Ill.	Jun. Assoc. M. M.	Oct. 3, 1893 June 1, 1898 Mar. 6, 1906
SMITH, WALTER MICKLE. Const. Engr., Ordnance Dept., U. S. A., Dover, N. J.	Assoc. M. M.	Oct. 2, 1901 April 3, 1906
THOMPSON, SANFORD ELEAZER. Cons. Engr., Newton Highlands, Mass.	Assoc. M. M.	April 1, 1896 May 1, 1906
TOMPKINS, EDWARD DE VOE. Cons. Engr. and Mgr., New York Office, Maine Elec. Co., 1 Madison Ave., New York City (Res., 372 Park Pl., Brooklyn, N. Y.)	Jun. Assoc. M. M.	Feb. 2, 1897 Dec. 3, 1902 May 1, 1906
WARD, THOMAS MONROE. Care, Canton Co., 15 South St., Baltimore, Md.		Mar. 7, 1906

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		Date of Membership.
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BOOZ, HORACE COREY. 663 Broad St. Station, Philadelphia, Pa.		April 4, 1906
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HEALD, EDWARD CRESWELL. Chf. Structural Engr., Office of Superv. Archt., Treasury Dept., 1720 N St., Washington, D. C.	Jun. Assoc. M.	Oct. 7, 1902 May 2, 1906
HENDERSON, ADELBERT ANDREW. 730 Franklin Ave., Wilkinsburg, Pa.		April 4, 1906
JEWEL, LINDSEY LOUIN. 504 Jeannette St., Wilkinsburg, Pa.		April 4, 1906
JUDELL, ADOLPH. Chf. Engr., Nevada Northern Ry., Toana, Nev.		April 4, 1906
KOLB, HENRY JACOB. Chf. Draftsman, Office of Chf. Engr., Brooklyn Rapid Transit System, 85 Clinton St., Brooklyn, N. Y.	Jun. Assoc. M.	Feb. 3, 1903 April 4, 1906
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PAWLING, GEORGE FRANKLIN. 1622 Real Estate Trust Bldg., Philadelphia, Pa.		April 4, 1906
PRENTICE, WILLIAM HENDRY, Jr. Asst. Engr., M. O. & G. R. R., Muskogee, Ind. T.		April 4, 1906
PRIEST, BENSON BULKELEY. Care, Am. Bridge Co., 42 Broadway, New York City		April 4, 1906
SANFORD, GEORGE OTIS. Asst. Engr., U. S. Reclamation Service, Williston, N. Dak.		Mar. 7, 1906
SINKS, FRANK FORREST. Vice-Pres., Condron & Sinks Co., 1442 The Monadnock, Chicago, Ill.		May 2, 1906
SMITH, CHARLES BAILEY. 1405 State St., Boise, Idaho.		Jan. 3, 1906
TURNER, HENRY CHANDLEE. 11 Broadway, New York City.		April 4, 1906
WAGNER, FRED J. Asst. Engr., Dept. of N. Y. State Engr., Sylvan Beach, Oneida Co., N. Y.		April 4, 1906
WILHELM, JEROME FREDERICK. Asst. Engr., Mo. Pac. Ry., Lock Box 181, Paragould, Ark.	Jun. Assoc. M.	Jan. 2, 1900 Mar. 7, 1906
WOOD, GEORGE ROY. Cons. Elec. Engr., Fulton Bldg., Pittsburg, Pa.		April 4, 1906
YATES, JOSEPH JOHNSON. 941½ South St., Elizabeth, N. J.		May 2, 1906

JUNIORS.

	Date of Membership.
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CROW, EDWARD. Harbour Board Office, Lyttelton, New Zealand	Oct. 3, 1905
DAY, WILLIAM PEYTON. 144 Sanchez St., San Francisco, Cal	Mar. 6, 1906
FISHER, CHESTER CENTENNIAL. U. S. Reclamation Service, Rupert, Idaho	April 3, 1906
FRYER, HENRY LE ROY. Aleda Apartments, Trenton, N. J.	April 3, 1906
GIBBS, ELBERT ALLAN. 302 Mitchell St., Ithaca, N. Y.	Mar. 6, 1906
HUBER, WALTER LEROY. 2120 Kittredge St., Berkeley, Cal	April 3, 1906
SHIPMAN, CHARLEY EVANS. Care, U. S. Reclamation Service, Billings, Mont	April 3, 1906
SOULÉ, EDWARD LEE. 608 Crossley Bldg., San Francisco, Cal	April 3, 1906

DEATHS.

SPIER, CHARLES LOUIS. Elected Associate, May 3d, 1905; died May 7th, 1906.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(April 8th to May 5th, 1906.)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Street Railway Journal*. New York City. Issues for first Saturday of each month 20c., other issues 10c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England, 25c.
(22) *Iron and Coal Trades Review*, London, England, 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Engineer*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, $1.
(29) *Journal*, Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l'Assoc. des Ing. Sortis des Ecoles Speciales de Gand*, Brussels, Belgium.
(32) *Memoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Genie Civil*, Paris, France.
(34) *Portefeuille Economique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Revue de Mecanique*, Paris, France.
(38) *Revue Generale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill., 10c.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussees*, Paris, France.
(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 8c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.

(54) *Transactions*, Am. Soc. C. E., New York City, $5.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $5.
(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Eng. Soc. W. Pa., 410 Penn Ave., Pittsburg, Pa., 50c.
(59) *Transactions*, Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 20c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 15c.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England.
(70) *Engineering Review*, New York City, 10c.
(71) *Journal*, Iron and Steel Inst., London, England.
(72) *Street Railway Review*, Chicago, 30c.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England.
(78) *Beton und Eisen*, Vienna, Austria.
(79) *Forschcrarbeiten*, Vienna, Austria.
(80) *Tonindustric-Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(82) *Dinglers Polytechnisches Journal*, Berlin, Germany.
(83) *Progressive Age*, New York City, 15c.

LIST OF ARTICLES.

Bridge.

Bridges. Willis Whited. (58) Apr.
A Flat Span Reinforced Concrete Bridge at Memphis.* (14) Apr. 7.
Rebuilding the Rondout Viaduct.* (14) Apr. 7.
The Distribution of Loads on Stringers of Highway Bridges Carrying Electric Cars. F. P. McKibben, M. Am. Soc. C. E. (13) Apr. 12.
Reinforced Concrete Bridges on the Chicago & Eastern Illinois.* (15) Apr. 13.
Curves for Reinforced Arches. Daniel B. Luten. (14) Apr. 14.
Steel Piling Cofferdams for Bridge Piers.* (14) Apr. 21.
The Pennsylvania Railroad Bridge at Havre de Grace.* (14) Apr. 28.
The Surface Finish of Concrete Bridge Masonry.* George S. Webster. (14) Apr. 28.
A "Double-Drum" Reinforced Concrete Arch Highway Bridge.* Daniel B. Luten. (13) May 3.
Die Illerbrücken bei Kempten im Allgäu.* D. Colberg. (From a Paper read before the Deutscher Beton Verein.) (51) Serial beginning Apr. 21.

Electrical.

Some Notes on Wires. Thomas Carter. (26) Mar. 30.
Recent Extensions at the Manchester Electricity Works.* (73) Serial beginning Mar. 30.
Portable Electric Tools and Their Industrial Application. Andrew Stewart. (73) Mar. 30.
Standardizing Rubber-Covered Wires and Cables. John Langan. (42) Apr.
Comments on Present Underground Cable Practice. Wallace S. Clark. (42) Apr.
Notes on Design of Hydro-electric Power Stations (With Reference to the Influence of Load Factor).* David B. Rushmore. (42) Apr.
The Relation of Load-Factor to the Evaluation of Hydro-electric Plants. S. B. Storer. (42) Apr.
Electric Motors for Driving Machine Tools. (21) Serial beginning Apr.
The Pennsylvania Railroad's Extension to New York and Long Island: Some Details of the Long Island City Power Station.* (14) Apr. 7; (40) Apr. 6; (18) serial beginning Apr. 14; (72) Apr.; (25) May.
The Nassau (L. I.) Light and Power Company.* (27) Apr. 7.
The Wiring and Maintenance of Shunt and Compound-Wound Motors. William Kavanagh. (27) Apr. 7.
An Example of House Lighting Design.* J. R. Cravath and V. R. Lansingh. (27) Apr. 7.
A Portable Selenium Photometer for Incandescent Lamps.* Theo. Torda. (73) Apr. 13.

* Illustrated.

Electrical—(Continued).

Electrical Equipment of Wanamaker's Philadelphia Store.* (27) Apr. 14.
The New Lighting and Power Station at Glenwood.* (14) Apr. 14.
Rates for Electric Current in Chicago. (Report to the Chicago City Council by B. J. Arnold and Wm. Carroll.) (14) Apr. 14.
Method of Design for Magnet Windings. F. Albert Willard. (27) Apr. 21.
Hydro-Electric Development in the Adirondacks.* (27) Apr. 21.
Speed Characteristics and the Control of Electric Motors.* Charles F. Scott. (9) May.
Installation and Maintenance of a Small Electric Light Plant.* (64) May.
Usine Elévatoire de Messein pour l'Alimentation de la Ville de Nancy en Eau Filtrée.* Mauduit. (33) Mar. 31.
Der Blitzschutz von Eisenbetonbauten. A. Kleinlogel. (78) Apr.
Vereinigte Schaltung und Bedienung von Betriebsmaschinen in Elektrischen Zentralen.* Karl Wertenson. (48) Apr. 14.

Marine.

On Vessels Constructed for Service in Our Colonies and Protectorates.* *Sir* Edward J. Reed. (Paper read before the Inst. of Naval Archts.) (11) Apr. 6.
Gas Engines for Ship-Propulsion.* J. E. Thornycroft. (Paper read before the Inst. of Naval Archts.) (11) Apr. 13; (12) Apr. 20.
The Overhead Wire Cableway applied to Shipbuilding. J. L. Twaddell. (Paper read before the Inst. of Naval Archts.) (11) Apr. 13; (22) Apr. 13.
The Introduction of Cranes in Shipyards.* Alexander Murray. (Paper read before the Inst. of Naval Archts.) (11) Apr. 13; (47) Apr. 14.
The Efficiency of Surface Condensers. R. L. Weighton. (Paper read before the Inst. of Naval Archts.) (11) Serial beginning Apr. 13.
French Armoured Cruiser *Ernest Renan*. (12) Apr. 20.
High-Speed Motor-Boats.* James A. Smith. (Paper read before the Inst. of Naval Archts.) (11) Apr. 20.
Le Paquebot *La Provence* de la Compagnie Générale Transatlantique.* A. Dumas. (33) Serial beginning Apr. 7.
Die Turmdeckdampfer *Queda* und *Wellington*.* W. Kaemmerer. (48) Mar. 31.

Mechanical.

Cement and Hydraulic Limes, Manufacture, Properties and Use.* E. Candlot. (67) Serial beginning Mar.
Coal Conservation, Power Transmission and Smoke Prevention. Arthur J. Martin, M. Inst. C. E. (29) Mar. 30.
An Improved Briquette-Making Machine.* (12) Mar. 30.
Endless Wire-Rope Drives. William Hewitt. (8) Apr.
Test of De Laval Steam-Turbine.* (8) Apr.
Applications of Gas Engineering to the Brick Industry. S. S. Wyer. (76) Serial beginning Apr.
The Pennsylvania Railroad's Extension to New York and Long Island: Some Details of the Long Island City Power Station.* (14) Apr. 7; (40) Apr. 6; (18) serial beginning Apr. 14; (72) Apr.: (25) May.
The New Kiln Firing Process at the Lawrence Cement Co.'s Siegfried Mill.* (14) Apr. 7.
Vertical Retort Settings. E. Körting. (Abstract translation of paper read before the Brandenburg Assoc. of Gas and Water Engrs.) (66) Apr. 10; (83) May 1.
Water-Gas Plant at Trieste.* (66) Apr. 10.
Allis-Chalmers Extensions: Additions to the West Allis Works, Milwaukee, Wis.* (20) Apr. 12.
A High-Pressure Gas Distribution System.* (13) Apr. 12.
Gas Making in Vertical Retorts.* (12) Apr. 13.
Steam Consumption of Winding Engines. (12) Apr. 13.
The Present and Future of Power Gas Plants.* (Paper read before the Eng. and Sci. Assoc. of Ireland.) (66) Apr. 17.
Briquetting of Fuels and Minerals: Description of the Zwoyer Fuel Company's Process and the New Jersey Briquetting Company's Plant.* G. J. Mashek. (20) Apr. 19.
The Prevention of Smoke. Albert A. Cary. (14) Apr. 21.
The New Mechanical Equipment at the Joseph Campbell Factory, Camden, N. J.* (14) Apr. 21.
The Mechanical Plant of the Ford Memorial Building, Boston.* (14) Apr. 28.
8e Exposition Internationale de l'Automobile du Cycle et des Sports.* (36) Serial beginning Dec. 25.
Les Turbines a Gaz.* L. Sekutowicz. (32) Feb.
Généralités sur les Moteurs et Spécialement les Turbines a Gaz.* J. Deschamps. (32) Feb.
Essais du Foyer et de la Chaudière Marcel Deprez et Verney.* L. Lecornu. (37) Mar.

* Illustrated.

Mechanical—(Continued).

Verbesserungen im Dampfkesselbetrieb durch Vermehrten Wasserumlauf.* (82) Feb. 24.
Die Heissluftmaschine mit Grosser Kompression. R. Wotruba. (82) Mar. 31.
Die Herstellung von Hydraulischem Kalk in Frankreich.* (80) Mar. 31.
Der Generator in der Zementindustrie.* C. Naske. (48) Apr. 7.
Ueber die Formänderung von Drahtseilen.* Hirschland. (82) Serial beginning Apr. 7.
Antriebsarten von Walzenstrassen. Franz Gerkrath. (Paper read before the Südwestdeutsch-Luxemburgische Eisenhütte.) (50) Serial beginning Apr. 15.

Metallurgical.

New Developments in Dry Air Blast.* A. Steinbart. (Paper read before the Technische Verein.) (22) Mar. 30.
Smelting Iron Ore by Electricity. Eugene Haanel. (Address before the Canadian Club of Toronto.) (62) Apr. 26.
The Lungwitz Process of Zinc Smelting. Fred W. Gordon. (16) Serial beginning Apr. 28.
Schwebetransporte in Berg- und Hüttenbetrieben.* G. Dieterich. (50) Serial beginning Apr. 1.
Ueber die Konstitution des Roheisens.* P. Goerens. (50) Apr. 1.
Die Berechnung des Hochofenprofils und Ihre Grundlegenden Werte.* Bernhard Osann. (Paper read before the Südwestdeutsch-Luxemburgische Eisenhütte.) (50) Apr. 15.

Military.

The Pressure of Explosions. J. E. Petavel. (Abstracted from the Philosophical Transactions of the Royal Society of London.) (11) Mar. 30.

Mining.

Over-winding in Hoisting Operations.* Robert Peele. (6) Jan.
The Pressure of Explosions. J. E. Petavel. (Abstracted from the Philosophical Transactions of the Royal Society of London.) (11) Mar. 30.
Deep Level Shafts on the Rand.* Arthur E. Pettit. (Paper read before the Inst. of Mining and Metallurgy.) (45) Serial beginning Apr.
Air Hammer Rock Drills: A new Type of Drill for Mining Work. E. A. Rix. (45) Apr.
Head-Frames: Some Notes on the Principles of Construction. W. R. Crane. (45) Apr.
Steam Consumption of Winding Engines. (12) Apr. 13.
Sinking and Tubbing at the Methley Junction Colliery. Isaac Hodges. (Paper read before the Midland Counties Inst. of Engrs.) (57) Apr. 20.
The Application of Direct Cementation in Shaft-Sinking. C. Dinoire. (Abstract of paper read before the Société de l'Industrie Minérale.) (68) Apr. 21.
Hydraulic Dredging.* F. Danvers Power. (16) Apr. 21.
The Honigmann Method of Shaft Sinking.* Adolf E. Hartmann. (16) Apr. 21.
Schwebetransporte in Berg- und Hüttenbetrieben.* G. Dieterich. (50) Serial beginning Apr. 1.

Miscellaneous.

The Commercial Organisation of Engineering Factories. Henry Spencer. (12) Serial beginning Mar. 30.
Classification of Engineering Expenditures. F. H. Newell. (14) Apr. 28.
The Metric System Fallacy. (10) May.
The Proposal to Force the Use of the Metric System. H. H. Suplee. (9) May.

Municipal.

The Cost of Brick Pavement in Helena, Mont. Charles W. Helmick. (14) Apr. 7.
Vitrified Brick for Paving Purposes. (14) Apr. 14.
Ueber den Griechischen Asphalt und Seine Technische Bedeutung.* A. Ch. Vournasos. (82) Mar. 31.

Railroad.

Alternating Current Electric Systems for Heavy Railway Service.* B. G. Lamme. (65) Mar.
The Latest Great Northern Engines. Charles Rous-Marten. (12) Serial beginning Mar. 30.
New Atlantic-Type Locomotives: London, Brighton, and South Coast Railway.* (47) Mar. 31.
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* Illustrated.

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The Powell-Potter System of Automatic Control of Trains. (18) Apr. 7.
The Pennsylvania Railroad's Extension to New York and Long Island: Some Details of the Long Island City Power Station.* (14) Apr. 7; (40) Apr. 6; (18) Serial beginning Apr. 14; (72) Apr.; (25) May.
The Work of Well-Drilling Machines on the Pennsylvania R. R. Low-Grade Freight Line. W. R. Hulbert. (13) Apr. 12.
Mallet Compound Duplex Locomotives for the Guayaquil & Quito Railway. (13) Apr. 12.
Reinforced Concrete Work at the New Railway Terminal Station at Atlanta, Ga.* (13) Apr. 12.
New Shops of the Missouri, Kansas & Texas Railway, Parsons, Kansas.* (40) Apr. 13.
Details of the A. C.—D. C. Locomotives for the New York, New Haven & Hartford Railroad.* (15) Apr. 13.
Effects of a Sleet Storm on Different Types of Third Rail Protection.* (15) Apr. 13.
Electric Locomotive for the New York, New Haven & Hartford Railroad.* (27) Apr. 14; (17) Apr. 14.
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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

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STREET TRAFFIC IN NEW YORK CITY, 1885 AND 1904.

By Clifford Richardson, Assoc. Am. Soc. C. E.

To be Presented September 19th, 1906.

In December, 1885, Francis V. Greene, M. Am. Soc. C. E., read a paper before this Society, entitled "An Account of Some Observations of Street Traffic,"* in which he presented certain data, in regard to ten of the large cities of the United States, collected in that year by the employees of the Barber Asphalt Paving Company. He also called attention to the desirability of having traffic measured systematically at frequent intervals, on a uniform system, in the leading cities of the world, for comparative purposes. Since that date no further observations have been made, as far as the writer is aware, in any of the cities considered by Captain Greene, either in the United States or abroad, with the exception of a count, made in 1896, of the number of vehicles traversing Fifth Avenue, New York City, at various points, before the removal of the granite block pavement from that street.

In November and December, 1904, additional data were collected, at the writer's suggestion, by employees of the Barber Asphalt Paving Company, on ten streets in New York City, for the purpose of determining the traffic carried by a number of representative streets at that time. The results of this count, as worked out by

* *Transactions*, Am. Soc. C. E., Vol. XV, p. 123.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

employees of the company, were presented not long after in the engineering journals in comparison with the data of 1885, and with data for Paris and London cited by Captain Greene. The weights attributed to the vehicles, by those working up these data, were made to include the horse or horses, but this was not done in calculating the tonnage in Captain Greene's paper, and has not been the practice abroad. It seems, therefore, that the comparisons made at that time were not correct. All the data having since then come into the writer's possession, it has seemed worth while to present them here, in considerable detail, for future reference, and to compare the results derived from them with those obtained by Captain Greene, when calculated on the basis used by him in 1885, although they have also been worked out on a basis which, it is believed, represents more accurately the traffic carried by the streets in 1904. The data were collected with somewhat greater detail in 1904 than in 1885, as is shown in Table 1.

The observations were made by two persons, an observer and a recorder with a talley sheet. They were taken on two days (with the exception of those on Fifth Avenue, which occupied three days), from windows overlooking the various streets. They covered a period of eleven hours, from 7 A. M. to 6 P. M., being that time of day in which the principal traffic is concentrated, or one hour less than that covered in 1885.

For the purpose of determining the tonnage carried by the various streets, the writer has assigned the following weights to the different classes of vehicles, from data furnished by several of the most prominent carriage and truck builders.

Vehicle.	Weight.
Cab	1 050 lb.
Carriage	1 575 "
Automobile, electric	4 000 "
Automobile, gasoline	2 325 "
Electric truck	5 000 "
One-horse delivery wagon	1 000 "
Two-horse delivery wagon	1 500 "
Omnibus (Fifth Ave.)	3 200 "
Omnibus (streets other than Fifth Ave.)	1 550 "
Trucks, 1 to 8 tons	2 000 to 16 000 "

On this basis, the total tonnage per foot of width of street has been calculated, excluding the width of car tracks, where these exist, because the main traffic, especially on asphalt, is confined to that portion of the street lying between the tracks and the curb. This information is given in Table 2.

From the observations of 1885, Captain Greene divided the traffic into only three classes: light weight (less than 1 ton); medium weight (between 1 and 3 tons); and heavy weight (more than 3 tons). For the purpose of comparison with the data of 1885, the observations of 1904 have also been calculated on this basis, using, however, more detailed data in regard to vehicles which weigh more than 3 tons, as those of 6 and 7 tons are not uncommon now in the streets of New York. The results obtained in this way are presented in Table 3.

Having the preceding data, a comparison of the traffic of 1885 with that of 1904 can be made satisfactorily only on Fifth Avenue, the other points of observation in the two years not being the same. This comparison is given in Table 4.

It appears that on Fifth Avenue, 5 460 vehicles passed the Worth Monument in 1885, and 6 300 passed over the granite pavement in 1896; whereas, in 1904, 12 068 were observed between Thirty-third and Thirty-fourth Streets, an increase of about 120 per cent. In 1885 Captain Greene estimated the average weight per vehicle as 0.68 ton. In 1904, according to the weights which have been assumed as correct, the average weight per vehicle was 0.82 ton, or 0.83 ton on Captain Greene's basis. In 1885, 91% of the vehicles weighed less than 1 ton, while, in 1904, 79%, a much smaller number, were of this character. The traffic on this street, therefore, while still composed largely of light vehicles, has increased to an enormous extent in eight years, and the average weight per vehicle to a very considerable extent. The tonnage per foot of width, on Captain Greene's basis of weight, has increased from 94 to 251, or to 247 on the writer's basis.

Reference to Table 2 shows that, with the exception of First Avenue, at Twenty-sixth Street, Fifth Avenue carries the heaviest traffic in the City of New York, although this traffic is composed of the largest percentage of light-weight vehicles. This fact, however, must be given a great deal of consideration, since a small

tonnage of heavy-weight vehicles will undoubtedly cause a greater deterioration of the pavement than a larger traffic of lighter ones.

In 1885 the traffic on Broadway, near Pine Street, amounted to 273 tons per ft. of width. No enumeration was made at this point in 1904, but at the two points where the count was made, between Eighteenth and Nineteenth Streets, on asphalt, and between Franklin and Leonard Streets, on granite block, the tonnage only reached 107.2 in the one case, and 65.5 in the other. These figures are on the Greene basis of calculation, which includes the car tracks in the width of the street, and which must be used for a comparison of 1885 and 1904 data. On the writer's basis of calculation, the tonnage at these two points amounts to 169.8 and 96.9.

These figures are exceeded by those for First, Fifth and Eighth Avenues, and for Fourth Street. It is quite surprising, therefore, to find that the Broadway traffic is not a heavy one, at the present time, as compared with that of several other streets in New York, while it has, very possibly, decreased rather than increased since 1885.

It is an interesting fact that the heaviest traffic which has been counted in New York is on First Avenue, at Twenty-sixth Street, and that it is largely made up of vehicles of a heavy class, averaging 2.18 tons in weight, only 26% being of less than 1 ton. It is also worthy of note that the pavement on this street is not of a form of construction calculated to withstand such a traffic, as it is not supported by a hydraulic concrete base.

For the purpose of comparing the traffic on the streets of New York with that upon streets in London and Liverpool, it seems worth while to present here certain data in regard to the latter, collected by Captain Greene, from the papers of Messrs. Hayward, Deacon, Howarth and Stayton. This information is given in Table 6.

It appears that the tonnage per foot run on some streets in London was far greater, even in 1873, than anything found in New York, reaching 412 on King William Street, a street of the same width as Fifth Avenue, where the vehicles average 1.06 tons in weight and 16 484 in number. On the other hand, on no street in London was the average weight of the vehicles as great as that on First Avenue. It must be remembered, however, that in London there is an enormous number of omnibuses which are not found

on New York streets, for instance, of the 10 776 vehicles passing along Piccadilly, 32% are of this description, and average 2 tons in weight, while only 56% are cabs, averaging ½ ton each. Notwithstanding this, the traffic on Fifth Avenue approaches in tonnage per foot run that on Piccadilly in 1873. Unfortunately, there are no data available for the latter street at the present time. The traffic, no doubt, has increased largely.

It has been the custom on the Continent, and, in some cases, in England, to classify the traffic according to the number of horses attached to the vehicles, or, as it is denoted in Paris, by the number of collars. For the purpose of comparing the traffic on New York streets with that on the streets of Paris, as recorded in a series of observations made in 1881 and 1882, the New York data have been calculated on the basis of the number of collars passing the different points, and this is shown in Table 5.

For comparison with these figures, Table 7 shows the traffic on prominent streets in Paris.

The preceding data show that the traffic in Paris in 1881 and 1882 was very much greater than that on the streets carrying the heaviest traffic in New York City at the present time. In fact, no traffic has been counted on any street in any city in the world which can equal that for the Rue de Rivoli in Paris in 1881, which at that time was nearly three times as great as that on Fifth Avenue to-day, the width of the two streets being practically the same. As in the case of some of the London streets, the great tonnage carried by this street is to be attributed to the large number of extraordinarily heavy omnibuses which traverse it, these vehicles being much heavier than those used in New York or London, and each counting for three collars. If to each collar a weight of 0.5 is assigned, the tonnage on this street would amount to 534 per ft. of width, which is nearly double that on Fifth Avenue.

As has been said, according to the method adopted on the Continent, in England, and by Captain Greene in 1885, no consideration was given to the weight of the horses in calculating traffic to which the pavements were subjected from the number of vehicles enumerated. Mr. W. G. Root, District Manager of the Barber Asphalt Paving Company, of New York City, in working out the data which were presented in the engineering journals in 1905, included the

weight of the animals as well as that of the vehicles, in arriving at a conclusion as to the traffic carried by the various streets. On this basis he obtained the results given in Table 8.

To the writer, the latter method of statement, if the estimate of the weight of the vehicle and animal is correct, seems much more desirable. From the point of view of one who has been employed for many years in the construction of pavements and in observations as to the causes of their deterioration, it has become evident that the latter is due much more to the impact of the horses' hoofs, especially in the case of asphalt pavements, than to vehicles rolling quietly over the surface; this, of course, with the understanding that the base supporting the pavement is sufficiently rigid to do so satisfactorily and prevent vibration in the wearing surface.

For these reasons it seems desirable to consider the data in Table 8 as representing the true traffic on the various streets in regard to which the data were calculated, and to suggest that, in any observations made in the future, due consideration should be given to the weight of the animal as well as to that of the vehicle.

TABLE 1.—TRAFFIC ON NEW YORK STREETS, 1904, 7 A. M. TO 6 P. M. AVERAGE NUMBER OF VEHICLES.

Street.		Cabs.	Auto-Vehicles.			One-horse delivery wagons.	One-ton trucks.	Carriages.	Omnibuses.	Two-horse delivery wagons	Trucks.							Total Number of vehicles.
			Elec. tric.	Gaso-line.	Electric trucks.						2-ton.	3-ton.	4-ton.	5-ton.	6-ton.	7-ton.	8-ton.	
5th Ave.,	33d to 34th Sts.	6 162	838	272	74	1 399	351	1 895	370	132	323	129	68	35	8	5	7	12 068
1st Ave.,	26th to 27th Sts.	185	14	7	25	1 141	1 381	10	1	122	1 332	582	351	277	180	91	241	6 037
8th Ave.,	34th to 37th Sts.	417	37	37	44	2 349	928	114	5	292	720	375	209	141	40	12	49	5 769
Broadway,	18th to 19th Sts.	1 005	134	58	40	1 132	402	252	3	54	332	178	93	52	20	6	10	3 771
4th St.,	Wooster St. to West Broadway.	229	38	21	13	829	884	71	2	56	589	283	122	114	62	8	76	3 397
Broadway,	Franklin to Leonard Sts.	392	55	44	32	1 743	243	80	2	362	207	90	16	5	1	3	2	3 277
10th Ave.,	22d to 23d Sts.	62	3	2	4	1 007	458	11	0	341	548	205	118	81	17	16	19	2 892
3d St.,	Mercer to Greene Sts.	24	2	2	3	513	444	6	0	46	250	132	53	51	20	4	24	1 574
2d Ave.,	34th to 35th Sts.	32	3	2	6	559	206	15	2	77	169	69	30	40	6	1	8	1 225
34th St.,	Broadway to 7th Ave.	97	7	15	45	395	148	52	0	49	116	45	15	19	2	0	13	1 018

TABLE 2.—TRAFFIC ON NEW YORK STREETS, 1904, 7 A. M. TO 6 P. M.

Street.		Width, in feet.	Vehicles.									Tonnage.			Collars.	
			Less than 1 ton.		From 1 to 3 tons.		From 4 to 6 tons.		From 7 to 8 tons.		Total.	Total.	Per vehicle.	Per foot of width.	Total.	Per foot of width.
			No.	%	No.	%	No.	%	No.	%						
5th Ave.,	33d to 34th Sts.	40	9 588	79.45	2 357	19.54	111	0.92	12	0.09	12 068	9 875.0	0.82	246.9	16 007	400.2
1st Ave.,	26th to 27th Sts.	44	1 555	25.76	3 342	55.86	808	13.38	332	5.50	6 037	13 167.8	2.18	299.2	10 150	230.7
8th Ave.,	34th to 37th Sts.	44	3 172	54.98	2 146	37.20	390	6.76	61	1.06	5 769	7 683.8	1.33	174.6	8 049	182.9
Broadway,	18th to 19th Sts.	25	2 443	64.78	1 147	30.42	165	4.38	16	0.42	3 771	4 244.3	1.47	169.8	5 033	201.3
4th St.,	Wooster St. to West Broadway.	32	1 185	34.88	1 830	53.88	298	8.77	84	2.47	3 397	5 772.0	1.70	180.4	5 091	159.1
Broadway,	Franklin to Leonard Sts.	28	2 577	78.64	673	29.53	22	0.67	5	0.16	3 277	2 714.4	0.83	96.9	4 143	147.9
10th Ave.,	22d to 23d Sts.	34	1 421	49.14	1 220	42.18	216	7.47	35	1.21	2 892	4 237.8	1.47	124.6	4 388	129.0
3d St.,	Mercer to Greene Sts.	28	589	37.42	833	52.93	124	7.88	28	1.77	1 574	2 469.0	1.57	88.0	2 263	80.8
2d Ave.,	34th to 35th Sts.	46	683	55.76	457	37.31	76	6.20	9	0.74	1 225	1 568.5	1.28	34.1	1 706	37.1
34th St.,	Broadway to 7th Ave.	24	593	58.25	376	36.93	36	3.54	13	1.28	1 018	1 251.1	1.23	52.1	1 415	58.9

NOTE.—The width does not include the car tracks, as these carry only a small proportion of the traffic.

TABLE 3.—Traffic on New York Streets, 1904, 7 a. m. to 6 p. m.

Street.		Width, in feet.	Vehicles. Less than 1 ton. No.	%	From 1 to 3 tons. No.	%	From 4 to 6 tons. No.	%	From 7 to 8 tons. No.	%	Total.	Tonnage. Total.	Per vehicle.	Per foot of width.	Collars. Total.	Per foot of width.
5th Ave.,	33d to 34th Sts..................	40	9 588	79.45	2 357	19.54	111	0.92	12	0.09	12 068	10 036.0	0.83	250.9	16 007	400.2
1st Ave.,	26th to 27th Sts................	44	1 555	25.76	3 342	55.86	808	13.38	332	5.50	6 037	13 017.5	2.16	295.9	10 150	230.7
8th Ave.,	34th to 37th Sts................	44	3 172	54.98	2 146	37.20	390	6.76	61	1.06	5 769	7 865.0	1.36	178.8	8 049	182.9
Broadway,	18th to 19th Sts................	25	2 443	64.78	1 147	30.42	165	4.38	16	0.42	3 771	4 287.5	1.14	171.5	5 033	201.3
4th St.,	Wooster St. to West Broadway	32	1 185	34.88	1 830	53.88	298	8.77	84	2.47	3 397	6 032.5	1.78	188.5	5 091	159.1
Broadway,	Franklin to Leonard Sts.......	28	2 577	78.64	673	20.53	22	0.67	5	0.16	3 277	2 752.5	0.84	98.3	4 143	147.9
10th Ave.,	22d to 23d Sts..................	34	1 421	49.14	1 220	42.18	216	7.47	35	1.21	2 892	4 259.5	1.47	125.3	4 388	129.0
3d St,	Mercer to Greene Sts...........	28	589	37.42	833	52.93	124	7.88	28	1.77	1 574	2 652.5	1.69	94.7	2 263	80.8
2d Ave.,	34th to 35th Sts....	46	683	55.76	457	37.31	76	6.20	9	0.74	1 225	1 622.5	1.32	35.3	1 706	37.1
34th St.,	Broadway to 7th Ave...:.......	24	593	58.25	376	36.93	36	3.54	13	1.28	1 018	1 283.5	1.26	53.5	1 415	58.9

Note.—The width does not include the car tracks, as these carry only a small proportion of the traffic.

TABLE 4.—Traffic on New York Streets: Comparison of 1885 and 1904.

Year.	Street.		Width, in feet.	Pavement.	Vehicles. Less than 1 ton. No.	%	From 1 to 3 tons. No.	%	Over 3 tons. No.	%	Total.	Tonnage. Total.	Per vehicle.	Per foot of width.
1885	Broadway,	near Pine St..............	40	Granite	4 379	54	2 502	34	930	12	7 811	10 905	1.39	273.0
	5th Ave.,	opposite Worth Monument	40	Asphalt.	4 957	91	373	7	130	2	5 460	3 744	0.68	94.0
1904	Broadway,	18th to 19th Sts............	40*	Asphalt.	2 443	64.78	1 147	30.42	181	4.80	3 771	4 287.5	1.14	107.2
	Broadway,	Franklin to Leonard Sts..	42*	Granite	2 577	78.64	673	20.53	27	0.83	3 277	2 752.0	0.84	65.5
	5th Ave.,	33d to 34th Sts............	40	Asphalt.	9 588	79.45	2 357	19.54	123	1.04	12 068	10 036.0	0.83	250.9

* In 1904 the width of the streets has been taken exclusive of that of the car tracks, as these carry but a small portion of the traffic. For the purpose of comparison with the data of 1885 the width of Broadway has been considered as including the car tracks in this table.

TABLE 5.—TRAFFIC ON NEW YORK STREETS, 1904, 7 A. M. TO 6 P. M., BY COLLARS.

Street.	Width, in feet.	Vehicles. One-horse. No.	%	Two-horse. No.	%	Three-horse and over. No.	%	Total.	Collars. Number.	Per foot of width.
5th Ave., 33d to 34th Sts	40	8 184	67.8	3 829	31.7	55	0.5	12 068	16 007	400.2
1st Ave., 26th to 27th Sts	44	2 714	44.9	2 534	42.0	789	13.1	6 037	10 150	230.7
8th Ave., 34th to 37th Sts	44	3 731	64.7	1 796	31.1	242	4.2	5 769	8 049	182.9
Broadway, 18th to 19th Sts	25	2 597	68.9	1 086	28.8	88	2.3	3 771	5 033	201.3
4th St., Wooster St. to West Broadway	32	1 963	57.8	1 174	34.6	260	7.6	3 397	5 091	159.1
Broadway, Franklin to Leonard Sts	28	2 422	73.9	844	25.8	11	0.3	3 277	4 143	147.9
10th Ave., 22d to 23d Sts	34	1 529	52.9	1 230	42.5	133	4.6	2 892	4 388	129.0
3d St., Mercer to Greene Sts	28	983	62.4	492	31.3	99	6.3	1 574	2 263	80.8
2d Ave., 34th to 35th Sts	46	799	65.2	371	30.3	55	4.5	1 225	1 706	37.1
34th St., Broadway to 7th Ave	24	655	64.4	329	32.3	34	3.3	1 018	1 415	58.9

1 Collar = Cabs, Gasoline Autos, 1-Horse Delivery Wagons and 1-Ton Trucks.
2 Collars = Carriages, Omnibuses, 2-Horse Delivery Wagons, Electric Trucks, Electric Autos, and 2, 3 and 4-Ton Trucks.
3 Collars = 5, 6, 7 and 8-Ton Trucks.

TABLE 6.—TRAFFIC RECORDS ON CERTAIN STREETS IN LONDON AND LIVERPOOL, 1873 TO 1884.

City.	Street.	Pavement.	Width, in feet.	Total number of vehicles.	Tonnage. Total.	Tonnage. Per vehicle.	Tonnage. Per foot of width.
London....	Gracechurch St....	Asphalt....	32	12 148	13 507	1.11	422
"	King William St...	Wood......	40	15 513	16 484	1.06	412
"	The Poultry.......	Asphalt....	22	8 167	8 330	1.02	378
"	Strand and Fleet St.	Wood......	37	16 208	13 596	0.84	367
"	Parliament St.....	Macadam..	45	14 306	14 380	1.01	322
"	Oxford St..........	Wood	57	16 886	17 076	1.01	300
"	Cheapside..........	Asphalt....	32	9 419	9 260	0.98	290
"	Leadenhall St......	"	30	6 128	7 588	1.08	253
"	Piccadilly..........	Macadam..	37	10 776	9 358	0.87	253
"	Euston Road......	Granite....	44	12 132	10 658	0.88	242
"	Brompton Road...	Wood......	....	....	....	....	216
"	King William St...	Granite....	32	6 371	6 506	1.02	203
"	Edgeware Road...	"	43	8 212	8 376	1.02	195
"	Regent St..........	Macadam..	52	10 796	9 668	0.90	186
"	King's Road.......	Wood......	....	....	....	....	156
"	Victoria St.........	Macadam..	40	6 040	5 780	0.96	145
"	Sloane St...........	Wood......	....	....	....	....	93
Liverpool..	....	Granite....	....	....	....	....	382
" ..	....	"	....	....	....	....	232
" ..	Great Howard St..	Wood......	....	....	....	....	231
" ..	Bold St............	"	....	....	....	....	100

TABLE 7.—TRAFFIC ON STREETS IN PARIS, FRANCE, 1881-1882, BY COLLARS.

Street.	Width, in feet.	Number of Collars.	Collars, per foot of width.	Total number of vehicles.
Avenue de l'Opera..........	52.48	36 185	689.5	29 460
Rue de Rivoli...............	39.36	42 035	1 067.9	33 232
Rue Saint-Honore..........	28.208	19 672	697.4	16 589
Boulevard Haussmann.....	45.92	14 096	306.9	12 638
Avenue des Champs Elysées	89.872	14 082	156.7	12 023

TABLE 8.—TRAFFIC ON NEW YORK STREETS, 1904, 7 A. M. TO 6 P. M., WEIGHT OF ANIMALS INCLUDED WITH VEHICLES.

Street.		Average tonnage per vehicle.	Average tonnage per linear foot of width for 11 hours.
5th Ave.,	33d to 34th Sts....................................	1.64	481.85
1st Ave.,	26th to 27th Sts....................................	3.18	435.58
8th Ave.,	35th to 36th Sts....................................	2.28	296.02
Broadway,	18th to 19th Sts....................................	1.97	299.63
4th St.,	Wooster St. to West Broadway.................	2.73	289.18
Broadway,	Franklin to Leonard Sts.........................	1.74	202.42
10th Ave.,	22d to 23d Sts......................................	2.69	227.45
3d St.,	Mercer to Greene Sts............................	2.51	140.50
2d Ave.,	34th to 35th Sts....................................	2.24	117.86
34th St,	Broadway to 7th Ave............................	2.03	89.22

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PAPERS AND DISCUSSIONS.

THE CONTROL OF HYDRAULIC MINING IN CALIFORNIA BY THE FEDERAL GOVERNMENT.

Discussion.*

BY MESSRS. H. H. WADSWORTH, FRANKLIN RIFFLE, J. D. GALLOWAY, H. DE C. RICHARDS AND STEPHEN E. KIEFFER.†

Mr. Wadsworth.

H. H. WADSWORTH, M. AM. SOC. C. E. (by letter).—Since the publication of Captain Harts' paper some additional data have been obtained relative to the hydraulics of the Yuba and Bear Rivers, and the movement of debris by them.

The construction of Barrier No. 1 has rendered possible a fairly close estimate of the quantity of water flowing in the Yuba River at various stages. A comparison of the cross-section of this dam with those of several forms of weirs, the coefficients of which in the usual weir formulas have been determined by several experimenters, gives an approximate value for its coefficient. During the high water of January, 1906, the depth of water on the crest of the dam was 5.3 ft. at the north end and 6.0 ft. at the south end. These depths indicate a flow of about 70 000 cu. ft. per sec., a determination which has been roughly checked by the United States Geological Survey at a measured cross-section near Smartsville.

During the flood of February, 1904, said to be the greatest for many years, the gauge height at The Narrows, about 6 miles above the barrier, is reported to have been 4 ft. higher than during the

* This discussion (of the paper by William W. Harts, M. Am. Soc. C. E., printed in *Proceedings* for February, 1906), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Presented at the meeting of the San Francisco Association of Members, Am. Soc. C. E., on April 14th, 1906.

flood of January, 1906. This extra depth means a discharge well up toward the 90 000 cu. ft. per sec., stated by the author to be the probable maximum. Mr. Wadsworth.

On the Bear River no recent flood discharge measurements have been made, but an extension of velocity and discharge curves, plotted from observations taken by the U. S. Geological Survey for several gauge heights, up to the maximum gauge heights observed, in connection with measured cross-sections, show that the assumed maximum flow for the Bear was nearly or quite reached during the flood of January, 1906.

The recent survey of the Bear River shows that conditions there have reached a state of comparative equilibrium.

DIAGRAM SHOWING THE VARIATION OF FLOW IN THE YUBA RIVER DURING THE FLOOD OF JANUARY 1906.

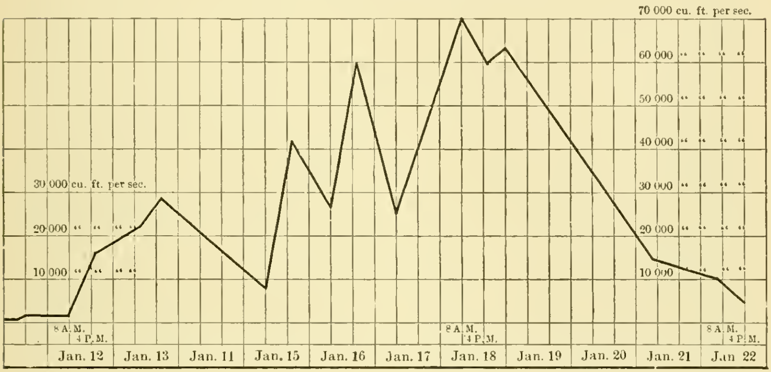

Fig. 4.

The form of the banks at the confluence of the Bear and Feather Rivers indicates that very little material is deposited in the latter by the former, and up stream, near Wheatland, the fact that alfalfa and hop fields are being cultivated between the levee lines shows that no great progressive damage is now being done from year to year.

On the Yuba, however, in spite of the fact that hydraulic mining on a large scale was stopped more than twenty years ago, there is still an extensive movement of the mining debris, as pointed out by the author, and this is likely to continue for many years. The great bulk of the material is moved during the few days of extreme high water each season. During the flood of January, 1906, the pool formed by the second step of the barrier was filled to the crest of the dam, at the center of its length, and the new deposit, extending up stream about 1½ miles, measured more than 920 000

Mr. Wadsworth. cu. yd. The continuance of a fairly high stage of the river since the flood mentioned has filled out the corners and increased the deposit to more than 1 000 000 cu. yd.

The filling in behind the first and second steps of the dam in such a short time is roughly confirmatory of the results of the survey, which showed a fill of 15 000 000 cu. yd. of material in the stretch of river bottom between Marysville and Barrier No. 1 in the five years between 1899 and 1904.

The diagram, Fig. 4, shows approximately the variation in the flow of the Yuba during a portion of January. This shows graphically the flashy character of the floods. The total discharge between 8 A. M. on January 12th and 4 P. M. on January 22d was approximately 1 000 000 000 cu. yd. of water. During this period was deposited the great bulk of the fill above the dam, which, from the foregoing figures, amounted to less than one-tenth of 1% of the mass of water by which it was transported. In addition to the material pushed along the bottom by the flood wave, a large quantity, of course, was carried over the dam in suspension. This may easily have amounted to one-third of the quantity deposited above the dam.

During the present season a spillway of sufficient capacity to carry the entire flow of the river for six months of the year will be built. As succeeding steps are added to the dam the spillway will be increased until, when the final height of the dam has been reached, its capacity will be such that water will flow over the crest of the dam only during extremely high water.

Besides the relief which this spillway will afford to the dam, its value as an accessory in the construction of succeeding steps will be very great, as the difficulties of turning the water and making the closures will be avoided.

Mr. Riffle. FRANKLIN RIFFLE, M. AM. SOC. C. E. (by letter).—The author has properly prefaced his excellent description of the methods used to control hydraulic mining in California by a brief historical review of this important industry. This feature adds materially to the value and interest of the paper. The development of hydraulic mining gave rise to many unique engineering problems which were boldly attacked and admirably solved by California engineers, whose achievements are but slightly known to civil engineers outside of that State. It is much to be regretted that an industry which has contributed so largely to the material wealth, not only of California, but of the entire country—which has placed in the hands of engineers hydraulic data of incalculable value—and is so distinctively an American enterprise—is not mentioned in the index of papers published in the *Transactions* of this Society. It is greatly to be hoped that some California member, familiar with the history of hydraulic mining, will rise to the occasion, and contribute a suit-

able paper on this subject before many important details of the early stages of the industry are lost to the engineering profession. Mr. Riffle.

The evolution from small volumes of water, low pressures, canvas pipe, and crude, improvised nozzles, to large volumes of water, high pressures, heavy double-riveted pipe of large diameter, and the hydraulic monitor or giant, took place in a surprisingly brief period, and, when taken in connection with the construction of ditches, flumes, tunnels, dams and reservoirs, often under unprecedented conditions, marks an important epoch in engineering history which deserves to be fittingly commemorated in the annals of America's leading society of civil engineers. It is well to remember that many of the great things accomplished by the engineers of to-day are to a very great extent the direct result of the experiments, mistakes, failures and successes of the engineers of yesterday. And so in California the engineers who were engaged in developing the industry of hydraulic mining more than a quarter of a century ago, through their failures and successes, have prepared the way for many of the spectacular achievements of the hydraulic and electrical engineers of the present day.

The author's statement that "the industry of hydraulic mining was finally completely vanquished," although evidently intended to apply to the region drained by the Sacramento and San Joaquin Rivers, may be misleading to those who are not residents of California. Hydraulic mining in California has never been prohibited by statute, although the decree of the United States Circuit Court which perpetually enjoined and restrained the North Bloomfield Mining Company from discharging mining debris into the Yuba River or any of its tributaries, had the effect of closing all hydraulic mines, the operation of which clearly caused injury to navigation or private property. In the northern part of the State, however, in the region drained by the Klamath River, hydraulic mining has been prosecuted without any interference whatever. Section 1424 of the Civil Code of California provides that:

"The business of hydraulic mining may be carried on within the State of California wherever and whenever the same may be carried on without material injury to the navigable streams or the lands adjacent thereto."

These conditions exist in the territory drained by the Klamath River. This, the second river in size in the State, is a typical mountain stream. It has a rapid, torrential current, and flows through a succession of deep gorges which extend almost to its mouth. There are practically no agricultural lands along its banks, and it is hopelessly unnavigable. High benches of auriferous gravel aggregating thousands of acres lie along its banks and those of its tributaries. Hydraulic mining operations have been conducted in this locality

Mr. Riffle. for many years with satisfactory results, the gravel yielding from $5 000 to $25 000 per acre. Although most of the mines are small, their aggregate yearly production of gold is very considerable. A few mines, however, operate on a large scale. The La Grange Mine, in Trinity County, for instance, has one of the largest hydraulic plants in the world. With high banks (the maximum height being 300 ft.) and 9 000 miner's in. of water available, it has been possible to hydraulic from 10 000 to 12 000 cu. yd. of gravel per day. To convey this large volume of water from the source of supply to the mine it was necessary to construct 16 miles of flumes, ditches, pipes and tunnels. This conduit contains a tunnel 1½ miles long, and a 30-in. riveted siphon nearly 1 mile in length operating under a pressure of 450 lb. per sq. in. at its lowest point.

Without going further into detail, it may be concluded that hydraulic mining promises to be an important industry in California for many years—especially in those localities which are more fortunately situated than the region described by the author.

Mr. Galloway. J. D. Galloway, M. Am. Soc. C. E. (by letter).—The engineers in charge of the construction of the Yuba barrier are to be congratulated on the success of their effort to build even a low dam across the Yuba River. Since the paper was written the river has been in flood, and, as no damage to the barrier has been reported, it may be considered that the second step in the dam will stand.

The writer is familiar with the Yuba River, and offers the following comment upon the subject of the paper. It does not seem, from the author's statements that any reliable estimate could be made of the quantity of debris now in the channels of the various forks of the river. Hence the ability of works, as designed, to control the situation, is open to question. According to the author, about 1 000 000 cu. yd. of earth per year are being mined, and this is restrained by log and brush dams. It is the writer's opinion that, in time, this will result in renewing the problem of the lower river. The brush and log dams will certainly decay, and, if upon any considerable stream, the debris will then be carried to the lower reaches and deposited. Owing to the fact that most of the tributaries are nearly or quite dry for a period every year, the decay of the dams will be rapid.

The head dam of the Bay Counties Power Company, 8 miles above Colgate, was formerly a wood-crib dam about 40 ft. high. It failed two years ago, after a service of some thirteen years, and was replaced by a stone dam. The writer examined the timber of this dam and found that, although it was in fair condition, it showed evidences of decay. Before failure the river was filled to the dam crest with debris, and of course, this went out with the dam. The plan of this dam was very much like the plan shown by the author,

and its failure is cited as an evidence that the dams built under the permits of the Debris Commission will fail in time, and in most cases the impounded material will be washed down the river. It is not quite as bad, however, as turning the debris directly into the stream, but, if the practice is continued, the condition of affairs will in time be nearly as bad as ever. Mr. Galloway.

Referring to the impounding of debris by the barrier, the author does not state the cubic contents of the basin above the dam, nor of the settling basins below. The quantity of debris brought down by the river is given as about 3 000 000 cu. yd. per year, but, without knowing the storage capacity of the basins, the ability of the works constructed to control the situation is not evident. A statement is made that after the first step was completed in 1904, the first freshet filled the basin to the crest of the dam.

The second step of the dam of course added a greater cube to the basin, and the author probably now knows what effect the late floods have had upon it.

Referring to the dam, the writer believes that the addition of two more steps, making it 36 ft. high, will be a serious menace to the entire structure. While part of the flood will be carried by the spillway, a relatively small quantity of water may do considerable damage to the toe. Water passing the series of rollways does not lose much of its energy, most of it being expanded in the pool at the toe.

The training walls from Daguerre Point to Marysville seem to the writer to be the best part of the scheme. Their value lies, not so much in lands reclaimed, as in preventing the river from washing the entire deposit of debris into the Sacramento River below.

The foregoing is not to be considered as a criticism on the design of the dam. It has been built on the shifting sands of a violent river, and the engineers are to be congratulated upon the success of the work. It is believed, however, that the building of temporary dams in the mountains to restrain debris is bad policy. Gold is not wealth, and the State would be better off if hydraulic mining had never been invented.

It is possible that the author has at hand the capacities of the basin above the barrier, intended to restrain the material of larger size, and of the settling basin for the finer material, in which case, an estimate of their ability to impound the debris could be made. Without this, it would seem that the project is open to question, as suggested above.

H. De C. Richards, M. Am. Inst. M. E. (by letter).—Mr. Hart's very able, comprehensive and highly educational paper refers to the protection for navigable streams developed on account of the great damage done by dumping mining debris into the Mr. Richards.

Mr. Richards. tributaries of those streams, but no reference is made to the debris from slides, which is steadily increasing and, in some sections, is much more serious than the mine dumpage.

The "Caminetti Law" so-called, has resulted less expensively in the long run for the miners than the fighting of many debris suits—some honest and many blackmailing.

The writer will describe briefly the conditions on the non-navigable streams of Northwestern California.

Several years' experience in hydraulicing gravels in Humboldt and Siskiyou Counties impels the writer to state that there are four counties in California where hydraulic mining can be done without injury to navigation or agricultural interests. The counties of Del Norte, Siskiyou, Trinity and Humboldt (north of Eureka) have an area of some 8 000 000 acres, or about 12 000 sq. miles.

The Klamath and Trinity Rivers, and the tributaries thereto having workable auriferous gravels within reasonable dumping distance, cover a length of about 600 miles. The lower 60 miles of the Klamath River has a fall of about 490 ft.

Placer mining began in that section about fifty years ago, and seems to have increased steadily in number of openings until some ten years ago, when many of the larger workings had reached the hill rock. The small "one-man workings" are innumerable; a few large mines are still operating on the Klamath, Trinity, Salmon and Scott Rivers, all of which, with the many smaller streams, flow to the Pacific through the Klamath. The lower 45 miles of this river has no large mines; the gravels are on the upper benches, some as high as 2 000 ft. above the river, with no water available for mining.

The mine now being operated by the writer at Orleans has averaged a clearing of about 1 acre of bed-rock per month with a full flume (5 by 3 ft., with a gradient of 10 ft. per mile) under a head of about 250 ft., and with banks from 50 to 80 ft. high. New workings, just opened, give 173 lb. static pressure on the gauge, that is, 402 ft. head. The same gauge indicates 120 lb. on the rear casting of a No. 4 giant, with a 6.5-in. nozzle, and with an overflowing penstock. There are several larger workings on that river, and the season is of about the same length, from 8 to 10 months—usually 8 months.

The whole country "slides easily." During the wet season, really phenomenal slides occur, stripping precipitous patches of bed-rock sometimes an acre or more in area and having depths of more than 100 ft. In one instance a length of about 1 500 ft., and more than 100 ft. thick, slipped into the Klamath River, completely damming it for some hours. Of course, much damage

was done a few hours later when it broke through. In the writer's opinion, the mining has not in the past and does not now put as much debris in the rivers as the slides in wet years. Mr. Richards.

Carefully kept rain gauges have recorded as follows:

1900-01	51.54 in.
1901-02	51.68 "
1902-03	59.30 "
1903-04	82.10 "
1904-05	44.27 "

In spite of these conditions, the rivers have not filled up, and, in fact, the Klamath and Trinity Rivers are so swift that they almost reach the normal scoured condition at any heavy flood season. Bars form and disintegrate from season to season, according to the stage of the water. The formation of a bar at any point of course indicates an increased jettying beyond.

Indian dugouts are the only craft the rivers float, and, aside from casual blackmail suits assisted by legal gentlemen on contingent fees, there is little to fear from legal restrictions in that section. The Honorable John F. Davis says that in these river basins the only foe with which the industry has to contend is the occasional blackmailer. The writer might add that the ranching which might be affected by the mining is ridiculously small, and that all the ranching land is nearly always more valuable for mining.

Many excellent quartz leads have been opened in that section, and a large number are mines in operation. Prospecting is more than encouraging in some of the newer districts, but is helped by the placer mining, and there are no complaints.

The following is from the Government Report on gold and silver production in 1904:

"Hydraulic mining property is in small demand in that area (drainage basin, San Joaquin and Sacramento Rivers), and few new mines are opened. Numbers have had their licenses withdrawn for violation of the rules of the commission granting them. Some changes in the laws will have to be made if hydraulic mining is ever again to flourish in this section.

"It is proper to note, and should be borne in mind by those interested, that these conditions exist only in the drainage area referred to. In all other parts of the State there are no restrictions on hydraulic mining. The extensive hydraulic operations in Siskiyou and Trinity Counties, and the smaller ones, in Humboldt and Del Norte Counties have always continued without interference by the people or the laws; and it is for this reason that Trinity and Siskiyou Counties now lead all others in this branch of mining. The drainage of all the streams in that area is ul-

Mr. Richards. timately into the Klamath River, which is a non-navigable stream, and empties directly into the Pacific Ocean, where all the debris finally disappears without possibility of damage to any one.

"Each mine may therefore dump its debris at convenience and the spring freshets carry it out, leaving a place for another dump the following year. Naturally, under these conditions (which formerly prevailed in the area of the State now under restriction), no impounding basins for tailings are necessary, nor is there any Federal supervision whatever. The quantity of gravel to be washed is therefore limited only by the quantity of water which can be brought to bear under pressure against the banks of gravel."

Mr. Kieffer. STEPHEN E. KIEFFER, ASSOC. M. AM. SOC. C. E. (by letter).— The author, in his analysis of the physical and industrial conditions leading to the cessation of hydraulic mining in the tributary water-shed of the Sacramento River, and the creation of the California Debris Commission, has clearly given the underlying reasons for the existence of the problem, and with the proposed method of its solution, that has proved too large for the State of California to cope with unaided, and too complex with bitter politically and socially to be dealt with in direct control by other than the Federal Government.

The bitterness engendered by the fight of a quarter century ago is a thing almost unknown to later comers in the State, especially if residence or business has not placed them in direct touch with the territory affected.

To-day, however, the feeling of animosity between the people of the valley and the people of the hills remains much the same as ever; the miners ever hope to mine and the ranchers in the valley jealously watch to see that the danger does not threaten.

As evidence of this fact may be cited a meeting of the ranchers, in the fall of 1904, at Marysville, called to consider the question of proceeding against the gold dredges at work along the Feather River at Oroville, and on the Yuba at Daguerre Point—the two latter dredges being those referred to by the author as building the lower diverting barrier across the Yuba.

Floods during the previous season had covered with sand large areas along the Feather River, and this was attributed to the disturbance of the gravel deposits in the rivers by the dredges, despite the fact that, of the forty dredges operating, only three were directly in the river channels and handling recent debris deposits.

When it was pointed out that the effect of the dredges was to cover the unstable surface sands and fine gravel with 30 ft. of coarse cobbles, thereby forever restraining the fine deposits from further movement in the stream, the agitation came to a sudden end.

At about the same time an attack was made upon the dredges of the American River field, the press of Sacramento claiming that the city pipes were filling with sand as a result of the dredges excavating gravel at Folsom, 22 miles distant. Mr. Kieffer.

These dredges were operating at a distance of ¼ mile from the channel of the American River, from 40 to 60 ft. in elevation above it, and tailed back into the river from ½ to 1 cu. ft. of water per sec. to each boat.

These cases serve to indicate how vigilant a watch is kept upon gravel mining in any form.

No other comment is needed upon the damage resulting to the valley lands from former hydraulic mining. While this vigilance guarantees immunity for the future, the writer feels that excessive alarm is unnecessary. To all intents and purposes hydraulic mining in the Sacramento water-shed is a thing of the past.

The author estimates that 1 000 000 cu. yd. of gravel are excavated per year at the present time. Taking his estimate of the yardage in the streams as follows:

Lower Yuba	333 000 000	cu. yd.
Bear	66 000 000	" "
Sacramento	108 000 000	" "
Other tributaries (assumed by writer)	100 000 000	" "

There is a total of 607 000 000 cu. yd. of gravel remaining in the streams.

If to this is added the *pro rata* of material estimated by the State Engineer in 1880 as being carried to Suisun and San Francisco Bays in suspension, the grand total is 760 000 000 cu. yd. of gravel mined and dumped into the streams in a period of about 25 years. No doubt the true volume is greatly in excess of this.

Upon this basis, it would take 760 years, at the present rate of hydraulicing, to cause a trouble of the magnitude confronting the Commission to-day, granting that all the gravel was allowed to go into the streams.

Comparatively few gravel deposits of the Sierras suitable for hydraulicing on any large scale have favorable sites for the storage of debris.

The necessarily small scale of the operations, combined with the added cost per cubic yard for restraining works, confines operations to the better paying or more favorably located gravels; hence, as long as the present regulations are in force, the field can never be greatly enlarged.

Granting that it is an open question as to whether or not the

Mr. Kieffer. type of restraining dams specified will prove a permanent barrier to the impounded debris, the writer cannot see that the occasional failure of a dam, in the course of time, can have any sigificant effect upon the problem as a whole, provided the general restrictive regulations of the present are enforced.

As far as possible, hydraulic mining should be encouraged. This the Debris Commission has apparently endeavored to do, but the results at best can be no more than a virtual wiping out of the industry in the Sacramento water-shed, thereby surely solving this phase of the problem.

The more momentous problem of what to do with the millions of cubic yards of debris already in the streams, however, is not as easily solved by restrictive measures. The Yuba project, so fully outlined by the author, cannot fail to impress one with the magnitude of the undertaking.

Although the Yuba is the worst offender, it is adapted to the plan of holding back the debris better than any other river of which the writer knows. Probably physical conditions could not be found elsewhere that would permit works of such magnitude to be constructed for such low cost.

Conditions are certainly unusual which will permit the Government to have built free of cost such an essential part of the scheme as the diverting barrier at Daguerre Point, with a height of 30 ft., a width of 200 or 300 ft. and a length of 2½ miles. Of the south training wall, 2 miles will also be constructed in a similar manner. The dredges building these banks excavate to a depth of 60 ft.

Even when gold values in the gravel of the river-bed fall below pay, as Marysville is approached, the construction of the south training wall should be effected at a very low figure by this method.

The great width of the Lower Yuba in its natural channel above Daguerre Point affords an unusual amount of storage for such a stream. Below Daguerre Point the width of 2 to 3 miles between levees would afford much greater room for subsidence outside of the training walls than is provided in the settling basin, although much of the most available part of this area on the south side will probably be mined by the dredges.

The elevation of the river-bed above the very flat valley lands makes the settling basin a very practical and easily accomplished part of the plan.

Barrier No. 1 is an interesting piece of engineering, the ultimate success of which seems to be assured. The length of the structure, its proposed height, the shifting character of the foundation, the floods to be passed, and the low cost for a virtually perma-

nent structure under the conditions, are all circumstances that will cause its future to be watched with interest. Mr. Kieffer.

The question of maintenance, in a structure with such an exposure of concrete face to the wear of passing gravel, may prove serious. The author states that during the first winter the concrete facing to a depth of 2 in. was cut out in places.

The writer has had experience along this line upon a dam built across the American River. The dam above the rubble masonry was crested with T-rails on each edge and filled, for a depth of from 6 to 8 in., to the tops of the rails, with concrete composed of 1 part cement and 2 parts river gravel passing a 1½-in. screen. This was entirely cut out during the first season; was replaced and cut out the next year, and finally replaced with 2-in. pine plank set between the rails.

The greatest damage from such erosion occurs during the middle stages of the river, when only the finer materials are being transported. That this danger is anticipated in Barrier No. 1 is indicated by the construction of a wasteway, 400 ft. long and 4 ft. deep, sufficient to carry the ordinary river flow. This will undoubtedly mitigate the trouble, but will not entirely remove it.

There is no doubt that the ordinary aggregate of the concrete will withstand this wear better than a strong mortar.

That the plan in general outlined by the Commission for the control of the debris of the streams is probably the most feasible one, there can be little doubt. How far it can be made beneficial is a question, and is largely a matter of topography and money.

The settling basin, diverting barrier and Barrier No. 1 of the Yuba project will probably impound from 40 000 000 to 45 000 000 cu. yd. of material. At the rate the lower Yuba basin filled, from 1899 to 1904, as given by the author, these works will be filled in about 10 or 15 years.

Other rivers, notably the American, afford poorer opportunity for impounding debris than the Yuba. In this case the river is frequently in flood when the Sacramento is not.

It is essential that all coarse material capable of being deposited in the slow-moving navigable streams should be held back if possible, but how far this can be done with the relatively small works capable of construction for the purpose only time will prove.

Vol. XXXII. MAY, 1906. No. 5.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NEW FACTS ABOUT EYE-BARS.

Discussion.*

By Messrs. Mace Moulton, John D. Van Buren and J. W. Schaub.

Mr Moulton. Mace Moulton, M. Am. Soc. C. E.—The author has stated that, under ordinary working stresses, the elongations in the shorter bars differ from those in the longer bars. Does it follow, therefore, that the general methods of computing the deflections will have to be modified on account of elements introduced by the difference in the lengths of the bars? This might occur, for example, in the case of a cantilever in which the top slopes toward the ends of the truss.

In the ordinary method of computing deflections, this difference in the lengths of the bars is usually taken into account, but, from the author's statement, it would seem that the deductions would be necessarily different in the case of bars of different lengths. As a result of the author's tests, is it possible to compute approximately how much allowance should be made?

Mr. Van Buren. John D. Van Buren, M. Am. Soc. C. E. (by letter).—While reading Mr. Cooper's valuable paper, it occurred to the writer that a rubber eye-bar would act in very nearly the same manner as a steel one, within the limits of elasticity, and would show the strains plainly on a small scale. The experiment was made, and the results illustrated by Fig. 11 are submitted at the suggestion of the author, who informs the writer that the lines of the rubber eye-bar correspond exactly in character with those of the large steel bars of his own experi-

* Continued from April, 1905, *Proceedings*. See January, 1906, *Proceedings* for paper on this subject by Theodore Cooper, M. Am. Soc. C. E.

Mr. Van Buren

EXPERIMENT WITH RUBBER EYE-BAR

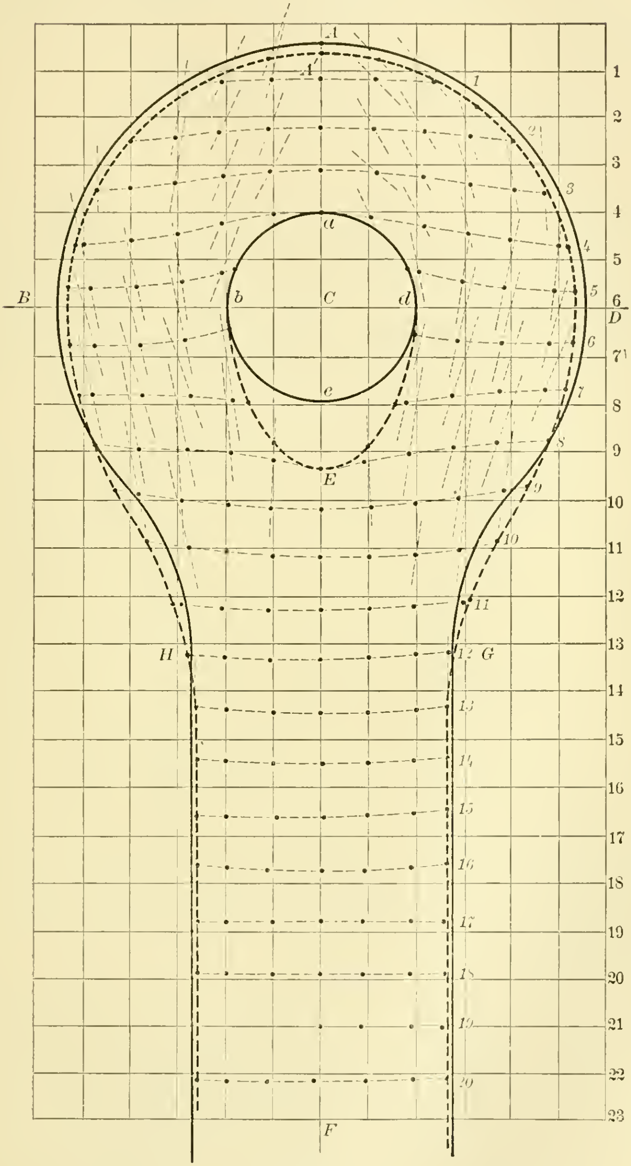

FIG. 11.

Mr. Van Buren. ment. The writer is in hopes that a more elaborate experiment with rubber may lead to results of some practical importance in determining the distribution of the stress quantitatively as well as qualitatively.

The dimensions of the eye-bar were as follows:

Neck $= 1\frac{3}{8}$ by $\frac{1}{2}$ in.;
Eye $= 2\frac{3}{4}$ in., outside diameter;
Pin $= 1$ in. diameter.

The bar, before the application of the stress, is shown in full lines, with squares inscribed on it. The bar, after the application of the stress, is shown in dotted lines. The pin-hole, as distorted by the stress, is shown by the dotted line, *a b E d*. The slanting dotted lines show the directions of the strains or flow. The small dots show the corners of the squares after distortion by the stress.

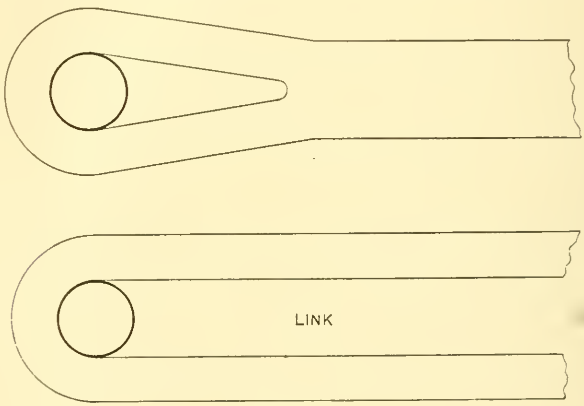

FIG. 12.

The difference between the areas of the original and the distorted squares, or between their sides or diagonals, measures approximately the stress at any particular place; tensile if the distorted squares, or lines, are the greater; compressive if they are smaller, with intermediate shearing stress.

The following indications are noted: The maximum stress is near the pin, along the lines, *b E* and *d E*, and is excessive, while the stress on the outer edges, *D* 8 and *B* 8, is comparatively small. The cause of this is evidently the bending action on each side of the pin just below *B D*, which increases the tension at the pin and reduces it on the outer edges. At *a*, for about half way to *A*, there is compression, and for the remainder of the distance there is tension; so that there is a neutral point between *a* and *A*. The top of

Mr. Van Buren.

the eye above line 4, or line 5, appears to be strained somewhat in the manner of a beam. There is compression at E, where the two streams of the flow meet. There is apparently a curved boundary of shearing stress starting near b and cutting $a\ A$ below A, and ending near d, surrounding the compressed area.

The excessive stresses and strains in the steel eye-bars, at and near each side of the pin-holes, account for the permanent elongations of the pin-holes even under moderate stresses, discovered by the author.

With a solid having considerable compressibility and a characteristic texture, however, the experiment is not complete without the measurement of the distortions in thickness, that is, in a direction perpendicular to the face of the bar, or diagram. As rubber has very little compressibility—in other words, has a very large modulus of elasticity of volume—it is evident that the volumes of the prisms represented by the squares will remain nearly constant, and that, therefore, the changes in areas of the squares will be accompanied by proportional changes, of an opposite character, in the thickness. Practically, with rubber, the coefficient of transverse linear expansion or contraction must be one-half that in a longitudinal direction—that of the load. The relations between the stresses and strains, in such a case, are comparatively simple; but, with more compressible solids having complex textures, it is difficult, if not impossible, to formulate this relation, even with a complete record of the distortions in the three directions. While it may not be possible to determine the stresses from the strains by the distortions in area alone, these distortions furnish a safe guide to the practical experimenter in search of the best shape—which requires the greatest possible uniformity in the stresses and strains. Progressive tests, marking the points of set and rupture, as carried on by the author, are the only safe guides. The mathematical theory of elasticity applied to a diagram of strains in a solid of complex structure leads to nothing practical.

Aside from considerations relating to the difficulties of manufacture, the results of this little experiment seem to point to the modifications indicated by Fig. 12 as remedies for the excessive stresses adjoining the pin. The excess in these shapes could be considerably reduced.

Mr. Schaub.

J. W. SCHAUB, M. AM. SOC. C. E. (by letter).—The results obtained by the author are not new. As he says, referring to his notes on the Eads Bridge, he finds, in pulling some iron eye-bars up to a proof stress of 18 000 lb. per sq. in., that a permanent deformation took place in the eyes. If the writer may be pardoned for the transgression, it may not be out of place to say that he believes the history of the building of the Eads Bridge to be the greatest educator in the art of bridge building, even to-day.

Mr. Schaub. In those days it was customary to pull all eye-bars, in the shop, up to a proof stress, usually twice the working stress, or about 18 000 to 20 000 lb. per sq. in. This applied to all iron eye-bars, and was the accepted practice until the advent of the steel bar. This test was for the purpose of developing any flaws in the head which might exist in the weld; but, as far as the writer knows, it never developed anything, except that a permanent set was produced in the head of the bar. This fact was well known, and in looking over his notes on iron eye-bar tests made at Edge Moor, in the period from 1881 to 1883, the writer finds in many cases the note that a permanent set took place in the head of the bar. This deficiency was first noted when the original mill scale and cinder began to flake from the head of the bar, sometimes back of the pin, but usually near the neck of the bar. The permanent set given to the bar was never as much as $\frac{1}{32}$ in., so that little or no attention was paid to this deficiency, and, as long as the bar did not ultimately fail in the head, it filled all the requirements.

These distortions are not confined to eye-bars. They will be found in compression members, as well as in tension members, and in riveted connections as well as in pin connections; in fact, in all cases where the stress is applied to a theoretical point in the member, and where insufficient means are provided for distributing this stress into the body of the member. This defect is inherent in all designs, more or less, and cannot be avoided without providing a sufficient amount of extraneous material in the connections to distribute the stresses properly, within the limits of elasticity of the materials.

In the case of eye-bars, the difficulty can be overcome to a great extent by thickening the heads; and, at the same time, making the heads elliptical, or longer, so as to increase the metal in front and back of the pin. This was the shape of the heads made on iron bars, before the circular head came into general use, and should never have been abandoned.

The criticism offered by the author as to the present form of eye-bar is just, and the defect should be remedied.

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PAPERS AND DISCUSSIONS.

THE ECONOMICAL DESIGN OF REINFORCED CONCRETE FLOOR SYSTEMS FOR FIRE-RESISTING STRUCTURES.

Discussion.*

By Messrs. F. P. Shearwood, Mansfield Merriman, A. H. Perkins, Langdon Pearse, C. B. Wing and John S. Sewell.

F. P. Shearwood, M. Am. Soc. C. E. (by letter).—Captain Sewell's advocacy of a system which eliminates as far as possible all uncertainties in this popular mode of construction will recommend itself to all. Mr. Shearwood.

Although the various formulas for computing the strength of beams do not differ greatly in their final results, the many discussions on the subject, by their very existence, prove that the distribution of the stresses (especially of the so-called shearing stresses) is still an unsolved problem, a clear conception of which is not yet reached.

Tests on specially prepared specimens, or on such as have been carefully watched during construction, do not by any means fully furnish the desired information, for until the stresses in a reinforced beam can be analyzed with at least the same degree of confidence in the result as is permissible in steel construction, this method must give occasion for skepticism.

The writer believes that tests, almost invariably, have been made for a single loading, and it is as yet probably unknown whether beams of this construction do not deteriorate under frequent load-

* Continued from April, 1906, *Proceedings*. See December, 1905, *Proceedings* for paper on this subject by John S. Sewell, M. Am. Soc. C. E.

Mr. Shearwood. ings. Many authorities seem to admit that the concrete in the reinforced or tension side of a beam is strained beyond its strength when the beam is loaded to its working capacity, therefore it follows naturally that it is more or less injured, and possibly serious harm will occur with frequent loadings.

From the very small deflections recorded under test loads on these beams it appears to be evident that the steel reinforcement is not performing the work for which it was designed, and therefore the concrete is performing a duty for which it is not safely capable, and, by being stressed repeatedly to its ultimate strength, it is likely to be fractured. How the destruction of the tension value in the concrete will affect its adhesive qualities should be ascertained before reliance is placed on the permanency of reinforced concrete.

A series of experiments to discover whether reinforced concrete can fail by fatigue, or to find out at what unit stress concrete can be strained when reinforced, without deterioration, would be a valuable addition to the useful knowledge on this subject.

Mr. Merriman. MANSFIELD MERRIMAN, M. AM. SOC. C. E. (by letter).—The author's method does not seem to be a satisfactory one for the design of beams, because his curve giving the distribution of stresses above the neutral axis was deduced from tests in which concrete was ruptured under compression. While his Equations 1 to 5 may apply very well to cases of the rupture of beams, they seem to be inapplicable to problems of design, since the stress-strain curve for rupture is very different from that which prevails when the concrete is stressed only to such values as are allowed by specifications. The unit stresses that appear in these formulas are those of the ultimate compressive strength of the concrete and the elastic limit of the steel. In designing a beam, however, the allowable compressive unit stress in the concrete should not be higher than about one-sixth of its ultimate strength, and the allowable tensile unit stress in the steel should not be higher than about one-half its elastic limit. To use the author's formulas for the design of beams, two methods may be used: first, to divide his f_c and t_s by factors of safety, in order that the working unit stresses may agree with those required by the specifications; or, secondly, to multiply the given maximum bending moment by a factor of safety. While both these methods are often used, it is maintained by the writer that both are illogical, and that neither of them leads to reliable results. When a beam is to be designed to carry a given bending moment under assigned unit stresses, the design should be made from formulas in which that bending moment and those unit stresses appear, and these formulas cannot be derived except from stress-strain curves which agree with those unit stresses. The object of establishing formulas for design is to determine the proportions of beams which shall

Mr. Merriman.

have the safe required unit stresses under the given bending moments. Such formulas cannot be expected to apply to cases of rupture, neither can formulas set up for cases of rupture be expected to give reliable results in designing.

The author's conclusion, that the greatest economy is secured in designing a reinforced concrete beam when the cost of the steel is equal to the cost of concrete above the steel bars, cannot be accepted by the writer. The author's investigation, starting with his Equation 10, seems to be incorrect, because it assumes the depth, d, to be a variable quantity, while his Equations 2 to 5 determine this depth for given values of the unit stresses. That is to say, the solution of these four equations, which are correct for the stress-strain curve adopted by the author, give the depth, d, of the beam, and the section area, a, of the steel per unit of breadth, in terms of the assumed unit stresses. Now, if these quantities are determined from the fundamental equations, it is certainly not in order to derive one or both of them later by supposing that they are variables, thus introducing an assumption which contradicts the fundamental conditions of equilibrium.

Another objection to the author's investigation is that he takes h, in Equation 6, as the constant number, 0.85, after having shown that its value ranges from 0.83 to 0.92. Now, a discussion of Equations 2 to 6 will show that this assumption fixes the steel section area at 1.13%; and, after the percentage of steel is thus fixed, it is difficult to see how it can later be made to vary with the relative costs of the steel and concrete.

There can be no doubt, however, that the proper design of a reinforced concrete beam involves the question of minimum cost. This, in the writer's opinion, is to be determined by selecting proper allowable values for the unit stresses in the steel and concrete. For the concrete the highest compressive unit stresses allowed by the specifications should be used, usually about 500 lb. per sq. in. for 1:2:4 concrete, and about 350 lb. per sq. in for 1:3:6 concrete. The tensile unit stress for the steel, however, cannot be arbitrarily assumed, but must be selected so as to make the cost of the beam a minimum, provided it be not greater than the highest value allowed in the specifications. If the tensile unit stress for the steel is taken high, the section area of the rods will be small and the beam will be deep; if it is taken low, the section area of the rods will be large and the beam will be shallow. The proper tensile unit stress to be used should be that which makes the total cost of steel and concrete a minimum.

When concrete is stressed in compression up to about 500 lb. per sq. in., the stress-strain curve is found to be closely a straight line, and hence a straight line should be used above the neutral axis

Mr. Merriman. in order to deduce formulas for designing. Under a small bending moment, there is also tension in the concrete on the lower side of the beam; but, under the maximum bending moment for which the beam is to be designed, these stresses are allowed to equal the ultimate tensile strength of the concrete, so that hair cracks occur. The entire tensile resistance of the concrete below the neutral axis, however, cannot be entirely overcome, but tension exists for a short distance below that axis, and should be taken into account in deriving formulas for the design of beams. The stress-strain curve for these tensile stresses is known to differ somewhat from a straight line, but, on account of the small area under tension, it may be considered as straight for a certain distance.

For 1:2:4 concrete the allowable compressive stress is about 500 lb. per sq. in., and the ultimate tensile strength about 250 lb. per sq. in. For 1:3:6 concrete the allowable compressive stress is about 350 lb. per sq. in., and the ultimate tensile strength about 175 lb.

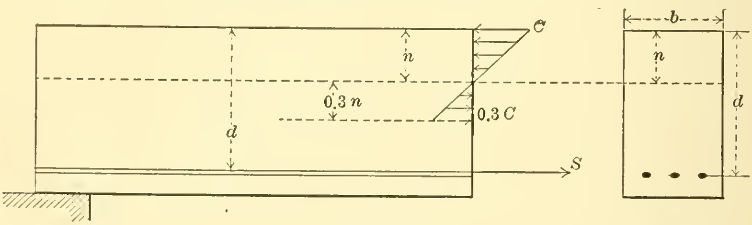

FIG. 32.

per sq. in. Hence the ultimate tensile strength of concrete is about one-half of the allowable compressive stress on the upper surface of the beam. Let C be the compressive unit stress allowed by the specifications for the upper surface of the beam, and n be the distance of the neutral axis below that surface; then, under a straight-line law, the greatest tensile unit stress in the concrete will be $\frac{1}{2}C$ at the distance, $\frac{1}{2}n$, below the neutral axis; but, on account of the known deviation of the stress-strain curve in tension from a straight line, it will be best to apply that law only as far as $0.3n$ below the neutral axis, thus making $0.3C$ the greatest tensile unit stress, and neglecting all tensile resistance lower than $0.3\,n$ from the axis. Fig. 32 shows, then, the internal stresses proper for consideration in deriving formulas for the design of reinforced concrete beams.

The equations of equilibrium for a rectangular beam of breadth, b, under a given bending moment, M, are now readily written. Let A be the section area of the steel rods, the center of which is at the distance, d, below the top of the beam, and let S be the tensile unit stress in these rods.

The first condition is that the sum of all the horizontal tensile stresses shall equal the sum of all the horizontal compressive stresses, whence, Mr. Merriman.

$$A\ S + \tfrac{1}{2}\ b\ (0.3\ n)\ (0.3\ C) = \tfrac{1}{2}\ b\ n\ C,$$

$$\text{or}\quad A\ S = 0.455\ b\ n\ C.$$

The second condition is that the resisting moment of all these stresses shall equal the bending moment. Taking the center of moments at the neutral axis, this condition gives

$$A\ S\ (d - n) + 0.455\ b\ n\ C\ (0.2)\ n + \tfrac{1}{2}\ b\ n\ C\ (\tfrac{2}{3}\ n) = M.$$

To these two conditions of statics must be added another which states the experimental fact that changes of length in horizontal lines on the side of the beam are proportional to their distances from the neutral surface. Let E_c be the modulus of elasticity of the concrete, and E_s that of the steel; then the shortening of a unit length of the upper surface of the beam is $\frac{C}{E_c}$, and the elongation of a unit length of the steel is $\frac{S}{E_s}$, and these are proportional to the distances, n and $d - n$. Accordingly,

$$\frac{C}{n\ E_c} = \frac{S}{(d - n)\ E_s},\quad \text{or}\quad \frac{d - n}{n} = \frac{S\ E_c}{C\ E_s}$$

is the third condition.

When a beam is to be designed, there are given its load and span, which determine the bending moment, M, the breadth, b, the moduli, E_c and E_s, and the allowable unit stress, C, for the concrete; usually, the allowable unit stress, S, for the steel is also assumed. Then the solution of the three equations above written will give the values of d, A, and n.

This solution furnishes the following formulas for designing beams in which, for the sake of abbreviation, the ratio, $\frac{E_s}{E_c}$, is represented by the letter, e:

$$d^2 = \frac{2.92\ (e\ C + S)^2\ M}{(e\ C + 1.33\ S)\ e\ C^2\ b},\quad A = \frac{0.455\ e\ C^2}{(e\ C + S)\ S}\ b\ d.$$

The first of these formulas gives the depth, d, and hence, also, the section area, $b\ d$, of the concrete above the reinforcing rods, while the second gives the section area of the steel. The determination of these two quantities constitutes the main part of the design of the beam when the breadth, b, is given.

In using these formulas to design a reinforced beam, the constant, e, is approximately known for each class of concrete. For all kinds of steel, E_s is closely 30 000 000 lb. per sq. in. For 1:2:4 concrete, E_c is about 3 000 000 lb. per sq. in., and hence e is about 10. For 1:3:6 concrete, E_c is about 2 000 000 lb. per sq. in., and hence

Mr. Merriman. e is about 15. The unit stress, C, should be taken as high as allowable by the specifications, in order to make the depth, d, as small as possible. As for the unit stress, S, it is also often customary to take the highest allowable value for steel given in the specifications, but it will now be shown by a numerical example that this practice leads to uneconomical design.

Let it be required to design a reinforced beam of 1:3:6 concrete, for which $\frac{E_s}{E_c} = e = 15$, the highest allowable unit stresses to be 350 lb. per sq. in. for the concrete, and 17 500 lb. per sq. in. for the steel. Let the breadth of the beam be 12 in.; the span, 14 ft.; and the uniform load, 300 lb. per lin. ft., including the weight of the beam. Hence, the maximum bending moment, M, is 88 200 lb-in. Using these data, Table 6 gives the depth, d, computed from the first of the above formulas for five different values of S, after which the

TABLE 6.

S, in pounds per square inch.	d, in inches.	$b\ d$, in square inches.	A, in square inches.	$b\ d + 60\ A$.	Relative cost.
7 500	11.2	134.4	1.18	205.2	107.2
10 000	12.1	145.2	0.80	193.2	101.0
12 500	13.0	156.0	0.59	191.4	100.0
15 000	13.8	165.6	0.46	193.2	101.0
17 500	14.6	175.2	0.37	197.4	103.1

corresponding section areas, $b\ d$ and A, are found. Each of these beams have the strength required by the specifications to carry the given bending moment with the assigned degree of security, but their costs are different. If the cost of the steel is 60 times that of the concrete, then the sums, $b\ d + 60\ A$ will be proportional to the costs for the five cases, and it thus appears that the selection of 17 500 lb. per sq. in., as the tensile working stress in the steel, produces dimensions which make the cost 3.1% more than when the stress is taken at 12 500 lb. per sq. in. Hence it is plain that the selection of the value of S is a matter of some importance.

Since both $b\ d$ and A may be expressed in terms of S, it follows that the value of S which renders the total cost a minimum is a problem of pure mathematics. Let p be the ratio of the cost of one cubic unit of steel to that of one cubic unit of concrete; then the value of S is to be obtained which renders $b\ d + p\ A$ a minimum, since $b\ d + p\ A$ is proportional to the total cost of that part of the beam above the center of the steel rods. For the sake of abbreviation, let r represent the ratio, $\frac{S}{C}$, or $S = r\ C$, so that S

is known as soon as r has been determined. Then the values of $b\ d$ and A are: Mr. Merriman.

$$b\,d = \frac{e + r}{\sqrt{e + 1.33\,r}} \sqrt{\frac{2.92\,M\,b}{e\,C}}$$

$$A = \frac{0.455\,e}{r\sqrt{e + 1.33\,r}} \sqrt{\frac{2.92\,M\,b}{e\,C}}.$$

Multiplying the expression for A by p, differentiating the sum, $b\ d + p\ A$, with respect to r, equating the derivative to zero, and solving for r, gives

$$r = 1.17\,\sqrt{e\,p} \ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{I}$$

as the value of r which is required in order that the cost of the beam shall be a minimum. Equation I is the first formula to be used in designing a reinforced concrete beam. For example, let $e = 10$ for 1:2:4 concrete, and $p = 60$; that is, let the cost of the steel per cubic unit be 60 times that of the concrete; then $r = 28.7$, or the unit stress, S, for the steel, must be 28.7 times as great as the unit stress, C, for the concrete.

After the ratio, r, has been ascertained from Equation I, the distance of the rods below the top of the beam is to be computed from

$$d = u\sqrt{\frac{M}{b\,C}}, \text{ in which } u = \frac{1.71\,(e + r)}{\sqrt{e\,(e + 1.33\,r)}} \ldots\ldots\ldots \text{II}$$

and then the section area of the steel is found by

$$A = v\,b\,d, \text{ in which } v = \frac{0.455\,e}{r\,(e + r)} \ldots\ldots\ldots\ldots\ldots\ldots \text{III}$$

These three formulas enable the design of a reinforced concrete beam to be made which will carry the given bending moment, M, with the required degree of security, and also be more economical than one of any other dimensions. If the question of economy is not considered, Equations II and III will furnish values of d and A for any assumed unit stresses, C and S, the ratio, r, to be used being the numerical value of $\frac{S}{C}$.

The distance of the neutral axis below the top of the beam can also be stated in terms of r, and also the ratio of the cost of the steel to that of the concrete, thus

$$\frac{n}{d} = \frac{e}{e + r} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{IV}$$

and

$$\frac{p\,A}{b\,d} = p\,v \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{V}$$

Mr. Merriman. from which the neutral axis can be located, and the cost of the steel relative to that of the concrete computed.

Tables 7 and 8 show the relations between the different quantities more clearly than can be shown by formulas. These tables have been computed for seven values of p, ranging from 30 to 90, this quantity, p, being the ratio of the costs per cubic unit of steel and concrete. Column 2 shows the values of r which must be used in order that the beam shall be of minimum cost, this ratio, r, being computed from Equation I, and being the ratio of the tensile unit stress, S, in the steel to the given compressive unit stress, C, on the upper surface of the concrete. Column 8 of Table 7 (for 1:2:4 concrete) gives values of S in pounds per square inch when C is taken as 500 lb. per sq. in., and Column 8 of Table 8 (for 1:3:6 concrete) gives values of S when C is taken as 350 lb. per sq. in.; these columns show that the unit stresses for the steel should decrease as steel becomes cheaper with respect to concrete.

Column 3 in Tables 7 and 8 gives values of u to be used in computing the depth of the rods below the top of the beam from Equation II. Column 4 gives the values of v for computing the section area of the steel from Equation III, and these numbers multiplied by 100 give the percentage of section area of the steel with respect to the concrete section area, $b\,d$; these columns show that the percentages of steel section to be used should increase as p decreases, and also that 1:2:4 concrete requires a slightly larger percentage of steel than 1:3:6 concrete. Column 5 shows the position of the neutral axis when the beam is stressed under the given bending moment; on the average, this axis is about 26% of the depth below the top for 1:2:6 concrete, and a little lower for 1:3:6 concrete.

TABLE 7.—FOR 1:2:4 CONCRETE. $\frac{E_c}{E_s} = e = 10$.

(1) Ratio, p	(2) r	(3) u	(4) v	(5) $\frac{n}{d}$	(6) $\frac{p\,A}{b\,d}$	(7) Approximate relative costs.	(8) Unit Stress, S, for C=500.
90	35	3.2	0.0029	0.22	0.26	100	17 500
80	33	3.2	0.0032	0.23	0.26	98	16 500
70	31	3.1	0.0036	0.24	0.25	95	15 500
60	29	3.0	0.0041	0.26	0.25	92	14 500
50	26	2.9	0.0049	0.28	0.24	89	13 000
40	23	2.8	0.0060	0.30	0.24	85	11 500
30	20	2.7	0.0076	0.33	0.23	81	10 000

Column 6 gives the ratio of the cost of the steel to that of the concrete, as found from Equation V, and this is seen to be not far from 25 per cent. Column 7, headed "Approximate relative costs,"

contains numbers which apply to reinforced concrete beams when properly designed by the method here presented. In computing these numbers, the allowable compressive unit stress, C, has been taken at 500 lb. per sq. in. for 1:2:4 concrete and at 350 lb. per sq. in. for 1:3:6 concrete, and no allowance for difference in cost between these two classes of concrete or for the extra concrete below the reinforcing rods has been made. Under this supposition, the cost of reinforced beams is about 10 or 11% higher when the lower grade of concrete is used. Mr. Merriman.

TABLE 8.—FOR 1 : 3 : 6 CONCRETE. $\frac{E_s}{E_c} = e = 15.$

(1) Ratio, p	(2) r	(3) u	(4) v	(5) $\frac{n}{d}$	(6) $\frac{p\ A}{b\ d}$	(7) Approximate relative costs.	(8) Unit Stress, S, for C=350.
90	43	3.0	0.0027	0.26	0.24	110	15 100
80	41	3.0	0.0030	0.27	0.24	108	14 400
70	38	2.9	0.0034	0.28	0.24	105	13 300
60	35	2.8	0.0039	0.29	0.23	102	12 300
50	32	2.7	0.0045	0.32	0.22	99	11 200
40	29	2.6	0.0055	0.34	0.22	95	10 200
30	25	2.5	0.0069	0.38	0.21	90	8 800

In Equation II, for computing the depth, a, the letter, C, appears, so that this formula may be used for any specified compressive unit stress. This formula, of course, is only one of many that may be deduced for different assumptions regarding the tensile stresses below the neutral surface, each assumption giving a different expression for u. While the common assumption, that the concrete below the neutral surface offers no tensile resistance, leads to somewhat different formulas for u and v, the numerical results obtained from them do not differ materially from those above given, as far as the values of u and v are concerned, although they give the position of the neutral axis somewhat higher. The writer has also worked out formulas and tables under the assumption that the tensile stresses in the concrete extend to the distance, $\frac{1}{2}n$, below the neutral axis, and finds that this gives the depths of beams and section areas of reinforcement about 3% greater than when no tension in the concrete is considered. Hence, neglect of the tensile resistances in the concrete is not on the side of safetly when beams are to be designed.

The writer's conclusions regarding the proper design of reinforced concrete beams are as follows:

1.—Formulas deduced from stress-strain curves of concrete tested to rupture in compression are irrational and unreliable for the design of beams.

Mr. Merriman. 2.—After the three fundamental equations have been written for any assumed stress-strain line, no further assumptions regarding the neutral axis or the lever arms of forces can be made without introducing contradictions which render the resulting formulas erroneous.

3.—When the unit stresses, C and S, are assumed, the three fundamental equations determine the depth of the beam and the section area of the steel.

4.—The practice of using for the steel a tensile stress as high as one-half the elastic limit leads to uneconomical design, unless the cost of steel per cubic unit is greater than about 90 times the cost of 1:2:4 concrete or greater than about 100 times the cost of 1:3:6 concrete.

5.—The unit stress, S, to be used for the steel rods, should be such that the cost of the reinforced beam shall be a minimum, and this value may be ascertained from Equation I.

6.—When no precise information is at hand, regarding the relative costs of steel and concrete, the value of r, in Equations II and IV, may be taken at about 31 for 1:2:4 concrete and at about 35 for 1:3:6 concrete. This gives the section areas of the steel as 0.36 and 0.39% for these two classes of concrete.

7.—Steel with a high elastic limit should not be used for reinforcing rods if its price per pound is higher than that of structural medium steel.

8.—Some formulas and tables in use require percentages of steel section area to an extent which is not only unnecessary, but wasteful and extravagant. The highest steel section area which may be used for economical design is 0.75% of the concrete area above the rods, and this is only allowable when the cost of steel per cubic unit is as low as 30 times that of the concrete.

9.—For economical design, the cost of the steel is about 25% of that part of the concrete which lies above the centers of the reinforcing rods. The use of steel to an extent which renders its cost equal to the cost of the concrete section, $b\ d$, leads to uneconomical design.

10.—A high-class concrete, for which the modulus of elasticity is about 3 000 000 and the allowable compressive stress 500 lb. per sq. in., is more economical for reinforced beams than a concrete having a modulus of 2 000 000 and an allowable compressive stress of 350 lb. per sq. in., unless the cost of the latter concrete is at least 10% less than that of the former.

Mr. Perkins. A. H. Perkins, Assoc. M. Am. Soc. C. E. (by letter).—That the moment equation for a reinforced concrete beam, as determined by any of the rational formulas for a given value of t_s, approximates very closely to a straight line, is a familiar fact to engineers work-

ing with curves such as those published in *Engineering News* by Mr. Perkins. Mr. Schaub as long ago as April 30th, 1903. There the moment curve is plotted with $\frac{a}{d}$ as ordinates and $\frac{M}{b\,d^2}$ as abscissas. That the curve with $\frac{a}{d}$ constant and t_s and $\frac{M}{b\,d^2}$ the varying elements also approximates a straight line, is new, and the resulting general equation is extremely simple. There is a general feeling among engineers, however, that what is wanted is not so much accurate formulas, although they are desirable, as accurate constants for use in formulas. Knowledge of proper values of E_c could hardly be in a more chaotic condition, and an extension of the knowledge of the proper percentages of reinforcement to be used with various mixtures would be received with pleasure by most engineers. However, even though the cart has been obtained before the horse, the possession of the cart is cause for congratulation.

That 0.8 represents the "reliability factor" of the strength of concrete, as compared with steel at its elastic limit, will be questioned by many engineers. The elastic limit of concrete is not more than three-fourths of its ultimate strength for the leaner mixtures, such as 1:3:6. This is shown clearly by the data presented by Professor Hatt* and by the Talbot experiments. With the richer mixtures, more common in reinforced construction, the elastic limit is probably a somewhat greater percentage of the ultimate strength. In addition, there is the variation in the strength of the same grade of concrete. It would seem, therefore, that it would have been better to have left the factor, 0.8, out of the equations, permitting the user to suit himself or the conditions in choosing the corresponding factor.

The writer observes that the author finds by his analysis the same position for the centroid of pressure that was obtained by Professor Talbot by analysis ($= \frac{7}{11}$ of y, above the neutral axis) at the time he reported his now classical set of experiments.

However completely a beam may be reinforced, it will fail as soon as the elastic limit of the metal is passed unless the percentage of reinforcement is below that which develops the full strength of the concrete at the elastic limit of the metal, for the reason that when the elastic limit of the metal is passed the neutral axis rises rapidly, and the unit stress in the upper fiber of the concrete increases. Hence, designing a beam that will not go to pieces when the elastic limit of the metal is passed, involves under-reinforcement. Web reinforcement is necessary, of course, under certain conditions, and perhaps desirable under nearly all conditions; however, it will not produce impossibilities, and should not be

* *Transactions*, Am. Soc. C. E., Vol. LIV, Part E, pp. 587 *et seq.*

Mr. Perkins. expected to produce a beam that will not go to pieces soon after the elastic limit of the metal is reached, if the percentages of reinforcement recommended by the author are used with ordinary mixtures. The elongation of 30 000-lb. elastic-limit steel at the elastic limit will be 0.1%, and between the elastic limit and the ultimate it will be about 25 per cent. For steel of 50 000-lb. elastic limit, the elongation at the elastic limit is 0.17% and at the ultimate 15 per cent. Now throw the author's Equation 4 into the form

$$k = \frac{n f c}{n f c + t_s}, \quad \text{where} \quad k = \frac{y_1}{d}, \quad \text{and} \quad n = \frac{E_s}{E_c}.$$

This is true for any shape of the stress-strain curve within the elastic limits of the materials, and may be used to find k at the ultimate by putting for t_s, 250 times t_s for steel of 30 000-lb. elastic limit, or 88 for steel of 50 000-lb. elastic limit (t_s remaining the stress at the elastic limit). It is evident that these values will give such attenuated values for k that the amount of reinforcement permissible would be "the ghost of a departed quantity."

TABLE 9.

t_s.	k.	$\frac{a}{d}$.	F.	$f_s(6M_c)$.	$\frac{E_s}{E_c}$.	Mixture.	$\frac{M}{bd^2}$.
30 000	0.4	0.0152	2 000	3 000	10	1 : 2 : 4	408
	0.333	0.0095	1 500	2 500	10	1 . 3 : 6	251
40 000	0.333	0.0095	2 000	3 000	10	1 : 2 : 4	339
	0.273	0.0058	1 500	2 500	10	1 : 3 : 6	211
50 000	0.286	0.0065	2 000	3 000	10	1 : 2 : 4	293
	0.231	0.0040	1 500	2 500	10	1 : 3 : 6	181
60 000	0.250	0.0048	2 000	3 000	10	1 : 2 : 4	260
	0.200	0.0029	1 500	2 500	10	1 : 3 : 6	159

This leads directly to the vexed question of factors of safety. If the concrete be reinforced up to the point where the steel reaches the elastic limit when the upper fiber of the concrete is at its ultimate, then the factor of safety of the concrete is only half that of the steel, a condition the reverse of what it should be, and that is manifestly a waste of metal. On the other hand, if $\frac{a}{d}$ be taken below the above-mentioned value, the beam goes to pieces before the full ultimate stress in the concrete is used. In other words, there is no way to utilize the factor of safety of 2 that steel has between the elastic limit and the ultimate strength. Of the two horns of the dilemma, it is clear to the writer that stressing the concrete below its ultimate is by far the short horn, and the better engineering.

Mr. Perkins.

The writer, therefore, would use, in the author's formulas, the values of $\frac{a}{d}$ and F shown in Table 9 modified by leaving out the factor, 0.8.

Perhaps a larger value of $\frac{E_s}{E_c}$, say 12, should be taken for the 1:3:6 mixture. This would make the resulting values of $\frac{a}{d}$ slightly larger. To the $\frac{M}{b\,d^2}$ in Table 9 the writer would apply a factor of safety of at least $2\frac{1}{2}$ for dead loads.

Mr. Pearse.

Langdon Pearse, Jun. Am. Soc. C. E. (by letter).—The writer has read Captain Sewell's interesting paper and the discussion thereon with much pleasure. He would like to add a few notes on the stress-strain curve of concrete in compression, based on tests made for the Boston Elevated Railway.*

These notes, made in connection with a thesis at the Massachusetts Institute of Technology in 1902, refer only to the 1:2:4 mixture. Plots of the stresses and strains were made on logarithmic paper, first the inelastic strains, that is, the strains produced by the first loadings, and second the elastic strains, that is the first strain minus the permanent set. Logarithmic plotting paper was used because of the well-known property that the logarithms of points on a parabola lie in a straight line.

An examination of the inelastic strain plots showed that in few cases could one straight line be passed through all the points plotted, whereas in many cases two straight lines at a considerable angle, and, in one or two cases, three straight lines could. This would mean that two or more parabolas might represent the curves. The elastic strains followed the straight lines more closely, so that one line was fairly representative for each set of observations.

In both cases the constants were derived for the range of stress from 300 lb. per sq. in. up to the ultimate strength, unless otherwise noted, assuming that the probable curve was $y^n = p\,x$,

where y = the stress, in pounds per square inch;

x = the change of length or strain, in inches per inch.

n and p are the constants to be derived.

The values used in the computations are given in Tables 10 and 11. The columns headed y_1 and x_1 give the low point, y_2 and x_2 the high point from which the constants, n and p, were calculated. The columns headed y_3 and x_3, and y_4 and x_4, give the points from which the values of n_1 and p_1 were determined. Tables 10 and 11, respectively, give the inelastic and elastic constants.

Using $y^n = p\,x$ as the stress-strain equation, according to the

* "Tests of Metals," U. S. War Dept., 1899.

Mr. Pearse.

TABLE 10.—INELASTIC STRESS-STRAIN CURVE FOR 1 : 2 : 4 CONCRETE.

Cement.	Time of Set.	y_1	x_1	y_2	x_2	n	p	y_3	x_3	y_4	x_4	n_1	p_1	Breaking strength, in pounds per square inch.	Remarks.
Alpha.....	7 d.	190	0.0001	300	0.0006	3.995	2.043×10^{13}	300	0.00062	700	0.0044	2.313	8.638×10^{8}	962	
Alpha.....	1 mo.	......	....	....	....	....	....	....	....	....	....	....	....	2 590	Points too scattered to draw curves.
Alpha.....	3 mo.	500	0.0001	2 360	0.001	1.484	1.011×10^{8}	....	....	....	....	....	....	3 226	
Alpha.....	6 mo.	430	0.0001	2 400	0.001	1.285	2.207×10^{7}	....	....	....	....	....	....	4 847	
Atlas	8 d.	350	0.0001	1 000	0.001	2.193	3.802×10^{9}	1 000	0.001	1 600	0.0051	3.469	5.22×10^{13}	1 695	
Atlas	1 mo.	420	0.0001	1 800	0.001	1.582	1.414×10^{8}	....	....	....	....	....	....	2 373	Curve breaks just below 1800.
Atlas	3 mo.	450	0.0001	2 100	0.001	1.495	9.236×10^{7}	2 100	0.001	2 900	0.001	4.294	1.845×10^{17}	2 918	
Atlas	6 mo.	400	0.0001	2 060	0.001	1.405	$4.5\ 7\times10^{7}$	2 400	0.0012	2 900	0.004	6.362	$2.66\ \times10^{24}$	4 000	
Germania.	7 d.	370	0.0001	1 240	0.001	1.904	7.759×10^{8}	....	....	....	....	....	....	2 400	
Germania.	1 mo.	......	....	....	....	....	....	....	....	....	....	....	....	2 933	Micrometer broke.
Germania.	3 mo.	500	0.0001	2 000	0.001	1.661	3.01×10^{8}	....	....	....	....	....	....	3 157	
Germania.	6 mo.	460	0.0001	1 940	0.001	1.600	1.82×10^{8}	1 940	0.001	3 000	0.0032	2.668	5.93×10^{11}	3 309	
Alsen.....	10 d.	335	0.0001	1 180	0.001	1.828	4.145×10^{8}	1 180	0.001	1 840	0.0043	3.283	1.22×10^{13}	1 862	
Alsen.....	1 mo.	......	....	....	....	....	....	....	....	....	....	....	....	2 373	Curve made of more than 2 parabolas.
Alsen.....	3 mo.	355	0.0001	1 560	0.001	1.555	9.262×10^{7}	....	....	....	....	....	....	2 600	Curve drops near end.
Alsen.....	6 mo.	420	0.0001	2 260	0.001	1.368	3.881×10^{7}	....	....	....	....	....	....	3 800	Curve drops.
Saylor's...	9 d.	184	0.0001	700	0.001	1.723	8.00×10^{7}	....	....	....	....	....	....	1 944	
Saylor's...	1 mo.	430	0.0001	1 300	0.001	2.081	3.023×10^{9}	1 300	0.001	2 100	0.006	3.736	4.296×10^{11}	2 119	
Saylor's ..	3 mo.	440	0.0001	1 800	0.001	1.635	2.093×10^{8}	1 800	0.001	2 600	0.004	2.770	1.866×10^{15}	2 778	
Saylor's...	6 mo.	500	0.0001	2 300	0.001	1.509	1.181×10^{8}	2 400	0.0011	4 200	0.0045	2.584	5.10×10^{11}	3 959	

definition of the modulus of elasticity, E, as the ratio of the change of stress to the change of strain, then Mr. Pearse.

$$E = \frac{d\,y}{d\,x} = \frac{p}{n\,y^{n-1}}.$$

This is the slope of the curve at the point, $x_1\ y$. Now if $y = 0$, $E = \infty$. This is clearly untrue, although for small values of stress the real strain is not known. It is probable that concrete is elastic, in the ordinary sense, up to 200 or 300 lb. per sq. in., and even up to 500 lb. per sq. in., in some cases, though permanent set is usually measured below that stress. The correct expression for the stress-strain curve in compression would seem to be a parabola with the vertex at zero, but with its axes slightly revolved so that the slope is finite at the origin, or else a combination of parabola and straight line—the straight line extending to the point at which permanent set is noted, the parabola from there to the point of ultimate strength.

TABLE 11.—ELASTIC STRESS-STRAIN CURVE FOR 1 : 2 : 4 CONCRETE.

Cement.	Time of set.	y_1	x_1	y_2	x_2	n	p	Remarks.
Alpha.......	7 d.	234	0.0001	820	0.0008	1.667	91 650 000	
Alpha.......	1 mo.	.....	....	.....	....	......	....	
Alpha.......	3 mo.	564	0.0001	2 840	0.0010	1.425	83 040 000	
Alpha.	6 mo.	422	0.0001	3 [illegible]00	0.0010	1.130	9 310 000	
Atlas.......	8 d.	395	0.0001	1 520	0.0010	1.708	273 300 000	
Atlas.......	1 mo.	425	0.0001	1 690	0.0010	1.668	242 000 000	
Atlas.......	3 mo.	515	0.0001	2 390	0.0010	1.500	116 800 000	
Atlas.......	6 mo.	442	0.0001	2 640	0.0010	1.289	25 580 000	
Germania...	7 d.	369	0.0001	1 820	0.0010	1.443	50 540 000	
Germania...	1 mo.	.....	....	.....	....	......	....	Micrometer broke.
Germania...	3 mo.	468	0.0001	2 600	0.0010	1.343	38 540 000	
Germania...	6 mo.	525	0.0001	1 840	0.0006	1.428	76 900 000	Curve flattens at top.
Alsen.......	10 d.	390	0.0001	1 800	0.0010	1.505	79 650 000	
Alsen.......	1 mo.	420	0.0001	2 090	0.0010	1.435	58 100 000	
Alsen.......	3 mo.	420	0.0001	1 630	0.0007	1.435	58 130 000	Curve flattens above this.
Alsen.......	6 mo.	570	0.0001	2 430	0.0008	1.434	89 600 000	
Saylor's...	9 d.	298	0.0001	1 560	0.0010	1.359	27 650 000	
Saylor's....	1 mo.	450	0.0001	1 800	0.0010	1.661	255 300 000	
Saylor's....	3 mo.	455	0.0001	2 190	0.0010	1.465	78 560 000	Curve flattens at top.
Saylor's....	6 mo.	580	0.0001	1 900	0.0005	1.356	56 020 000	

For all practical purposes, the writer favors the formulas based on a straight-line stress-strain curve, using constants based on working values of the fiber stresses in steel and concrete, neglecting the concrete in tension.

C. B. WING, ASSOC. M. AM. SOC. C. E. (by letter).—In a recent paper before the Canadian Society of Civil Engineers, Henry Goldmark, M. Am. Soc. C. E., has stated that most of the formulas proposed for the design of reinforced concrete are based on the Mr. Wing.

Mr. Wing. common theory of flexure of homogeneous materials, with modifications due to the composite nature of the beam and physical properties of concrete differing from those of steel.

A more accurate statement would be that proposed formulas may be divided into two classes, one class for which Mr. Goldmark's statement is true, and another class in which the modifications and assumptions are of such a character that all semblance to the ordinary theory of flexure is lost, and the resulting formulas are purely empirical.

The formula proposed by the author is of this latter class; and, for low percentages of reinforcement, with material of low elastic limit, will be found to give ultimate strengths agreeing closely with the results of tests.

The range of application of the formula, however, is limited, as will be shown later, and in inexperienced hands may give extremely dangerous results.

The stress-strain diagram of a reinforced concrete beam shows two critical points: the point of failure of the concrete in tension, and the elastic limit of the steel.

These points may be compared with the elastic limit and the point of rupture of the stress-strain diagram of steel in tension, with the difference that, in the case of steel, stresses beyond the elastic limit cause permanent deformation, while, in the case of reinforced concrete, stresses beyond the tensile strength of the concrete only lead to the opening up of cracks which are closed on the removal of the load if the elastic limit of the metal has not been exceeded.

The whole question of proper methods of designing reinforced concrete would seem to hinge on this one point, that is, to what extent it is safe to allow tensile cracks to form in reinforced concrete beams.

This point can only be settled satisfactorily by tests for the effect of corrosion and repeated stress on reinforced concrete beams in which such cracks have formed.

Until such tests have been made, conservative design would require that the maximum stresses in the outer fiber of reinforced concrete beams be kept within the limits of the ultimate tensile strength of the concrete, say 300 lb. per sq. in. for a fair quality of concrete.

If this principle is accepted, the best type of formula to use is one based on the ordinary theory of flexure.

At present the design of beams by such formulas is comparable to the condition that would exist if all tables of moments of inertia and properties of steel beams were to be destroyed.

However, by adopting a proper notation, and preparing tables,

it is possible to solve theoretical composite beam formulas with the same ease that solutions of formulas for homogeneous beams are obtained. Mr. Wing.

Such theoretical formulas can be shown to give results agreeing closely with the results of tests, both at the point of failure of the concrete in tension and at the point at which the elastic limit of the steel is reached.

This being the case, there would seem to be no justification for adopting a formula theoretically open to criticism, of limited application, and which, in inexperienced hands, would give dangerous results.

Table 12 has been prepared in order to show the difference in designs obtained by using the author's formulas with $t_s = 16\,000$ lb. per sq. in., and by using a formula based on the ordinary theory of flexure limiting the tensile stress in the concrete to 300 lb. per sq. in.

The beams in Table 12 have been calculated as having a resisting moment of 120 000 in-lb. The beams calculated by the ordinary flexure formulas are square, with the center of the steel reinforcement 1 in. from the lower surface of the concrete. The beams calculated by the author's formula are square, above the center of the steel, and have 1 in. added to the depth to provide a protective coating for the steel.

The cost of concrete was assumed at 20 cents per cu. ft., and the cost of steel at 3 cents per lb.

The author states that "the actual working stress in the concrete would seem to be of secondary importance as long as the factor of safety is assured." It is difficult to understand how the factor of safety is assured when beams designed by the author's formula show probable values of the compressive stress in the concrete as given in Table 12. These values have been calculated by the ordinary theory of flexure, neglecting the tension in the concrete, and may be higher than the stresses actually existing, but beams calculated by such formulas are on the side of safety.

The author, in justification of an empirical formula, speaks of the plate girder as a case in which the designer departs from the ordinary theory of flexure, but fails to state that such departure leads to the design of heavier beams than would be required by the ordinary theory of flexure.

It is to be regretted that the author's proposed formula and other similar formulas do not in all cases in like manner give results departing from the ordinary theory of flexure on the side of safety. It is safe to say that, if such were the case, technical literature would be but little burdened with discussions of the true form of the stress-strain curve for concrete in compression, and the old-fashioned Hooke's Law would be considered near enough for practical purposes.

Mr. Wing. The portion of the paper devoted to the determination of the economic percentage of reinforcement depends entirely upon the acceptance of the proposed formula. If that is not accepted, the whole argument is without foundation, and that the acceptance of this formula may lead to dangerous designs is indicated by the results given in Table 12, which show that beams designed by the ordinary flexure formulas will increase in cost with the percentage of reinforcement.

For economy, the percentage of reinforcement, therefore, should be as small as good practice will warrant, and is not a subject for theoretical determination.

TABLE 12.

Ordinary Flexure Formula.					Author's Formula.		
Percentage of reinforcement.	Side of square beam, in inches.	Cost per foot, in cents.	Probable maximum stresses per square inch, if concrete fails in tension.		Depth of beam; width 1 in. less.	Cost per foot, in cents.	Probable compression in concrete per square inch.
			Tension in steel.	Compression in concrete.			
½	12.8	31.0	14 300	530	12.4	26.1	640
¾	12.6	33.6	10 000	490	11.0	22.8	800
1.0	12.3	36.1	8 100	470	10.1	21.1	1 040
1¼	12.1	38.3	6 800	460	9.4	19.8	1 130
1½	11.9	40.5	6 000	460	8.9	19.2	1 280
1¾	11.7	42.6	5 500	460	8.5	18.9	1 440
2.0	11.5	44.7	5 000	460	8.2	18.6	1 560

Captain Sewell. JOHN S. SEWELL, M. AM. SOC. C. E.* (by letter).—After reading the discussion which has been brought out by his paper, the writer feels that its main object, which was to arouse a greater interest in certain points pertaining to the design of reinforced concrete, has been largely accomplished. He desires to express his gratitude for the kindly expressions of appreciation on the part of so many of those taking part in the discussion.

In order to set at rest the doubts apparently existing in the minds of some, he desires to say that he claims no originality for anything fundamental in any part of the formulas discussed or proposed by himself. Even the treatment of web stresses, which is different from any he has seen elsewhere, could not be called original, in the fullest sense of the word, and it may not be so, in any sense. The question of originality, however, is entirely secondary to that of correctness. The writer, in common with many other engineers, has studied such data as he had available, and, in the light of such study and his own experience, has deduced conclusions in which he has great personal confidence. But it is

*Captain, Corps of Engineers. U. S. Army.

realized that these conclusions are rather matters of opinion than thoroughly demonstrated facts, and it is hoped that the extensive series of tests about to be inaugurated under the auspices of this and other societies may cover the ground in so thorough a manner as to leave no room for further doubt or discussion. Captain Sewell.

The point raised by Mr. Jonson, in reference to taking the depth of the horizontal reinforcement below the top of the beam as a basis for the shear computation, is well taken. Referring to page 650,* and to Fig. 4, the depth should be $h\ d$, instead of d. The writer had intended to correct this error, but will content himself by acknowledging the accuracy of Mr. Jonson's criticism. It should, perhaps, be further explained, that Fig. 4 and its accompanying text constituted a demonstration of the writer's method of treating the web stresses, rather than a statement of the method itself. As a matter of fact, when the writer uses diagonal web members, if a be the area of the horizontal reinforcement, the aggregate section of the web members in one-half of the beam is taken as $\frac{1}{2}\ a\ \sqrt{2}$; if vertical web members are used, their aggregate area in one-half of the beam is taken as a; neither of these expressions would be quite correct if the depth were taken as d; nor would they result from an actual determination of stresses in a multiple-intersection truss with a finite number of web systems. The greater the number of web systems, the more nearly does the aggregate of the tensile web stresses approach equality with the aggregate of the compressive stresses; and, in a solid beam, they become equal; under these conditions, the writer's expressions for the aggregate section of web members are correct. If diagonal members are used, the length of each member is equal to $d\sqrt{2}$. The total volume of the members in one-half of the beam is then equal to $a\ d$. The same expression holds for the volume of the vertical members, so that the total weight of the web reinforcement is the same in the two cases, assuming the web members to extend to the top of the beam, in either case. In any case, the number of web members necessary to make up the aggregate section is determined by dividing the aggregate by the area of one member. Their spacing is determined in much the same manner as that used for the rivet spacing for a plate girder.

Replying to Mr. Watson: the unreasonable and illogical requirements of many building laws constitute one of the reasons for writing this paper. These laws often, on the one hand, put reinforced concrete at an unfair disadvantage, and, on the other, open up the way to very real dangers in design. The writer is also opposed to the use of high-carbon steel, or any steel with an elastic

* *Proceedings*, Am. Soc. C. E., for December, 1905.

Captain Sewell. limit exceeding, say, 40 000 to 45 000 lb. per sq. in. If the modulus of elasticity could be increased with the elastic limit, the matter might be different; but, as it is, the greater strength of the steel with a high elastic limit can be utilized only by permitting deformation beyond a reasonable limit, and by permitting the neutral axis to rise so high in the beam that the economy of the greater stress in the steel is largely lost, because of the greater quantity of concrete needed to counterbalance the steel stresses. The strength of the steel with a high elastic limit is also likely to be seriously impaired in a fire, which is in itself a sufficient reason for using soft or medium steel in fire-resisting structures, not to mention the other very valid objections to high-carbon steel urged by Mr. Watson.

Vertical stirrups, merely passing under the main bars, cannot possibly transmit any tensile stress into those bars, for no force has a component at right angles to itself; nor can such stirrups afford an abutment for the diagonal compressive stresses in the concrete, except in so far as the concrete itself serves as an anchorage for them. As one cannot lift himself by his own boot straps, it is difficult to know how the stirrups described by Mr. Watson can assist in any way except by reinforcing the concrete against local failure, and thus holding adhesion up to its full value. This seems to be only a partial solution of the real problem.

Replying to Mr. Noble: the writer expects that well-conducted experiments will prove quite conclusively that percentages of reinforcement, considerably greater than those deduced from his own equations, can be safely used. The writer, in assuming his constants, tried to keep well within safe limits, and the discussion indicates that in this, at least, he was successful.

The great value of the adhesion, and of the shearing strength in concrete is not denied; but both are subject to at least as many uncertainties as the tensile strength, and the writer prefers to let them all go into the indeterminate part of the factor of safety—especially in structures likely to be damaged by fire.

Mr. Kreuger's comments, concerning the width of flange in **T**-beams, touch upon a very interesting point—one which, as it is treated in such building laws as those of New York City, opens up the way to some decidedly dangerous designs. The writer's width of flange was deduced from a discussion based on the shearing strength of concrete. While he now disregards this, as far as possible, the width of the rib, in a **T**-beam, results from considerations affecting the transfer of the tensile stress from the steel into the compressed part of the concrete; the questions of how much stress can be distributed into the slab on either side, and how far it must go before being absorbed, are not usually easy to determine. But, the length of the span, and all other real factors, enter, in-

directly, into the design of the rib, and a width three times as great as the width of the rib will usually be found quite safe and sufficient for the flange, except in the rare case where very heavy beams are spaced very closely together, with a very thin floor slab on top of them. In that case, the thickness of the flange would seem to be the correct basis for determining its width. The writer sees no objection to making it the basis in all cases. Captain Sewell.

Referring to Mr. Dana's comments on the writer's reasons for using attached web members because of the facility for repairing beams damaged by fire, it should be pointed out that exposure of steel members results from the spalling of the concrete, rather than from complete dehydration of the cement. The Baltimore fire, and many fire tests have demonstrated that the steel itself is often exposed, without suffering serious damage; it is to be presumed that the fire is usually about exhausted, in such cases, before the concrete finally comes off. But, for fire-resisting structures, the writer is opposed to steel of very high elastic limit, because of the very danger pointed out by Mr. Dana. If every beam that has its main rods exposed has to be torn out and rebuilt, reinforced concrete will not long be in favor with underwriters, for the salvage will hardly be greater than in the case of a timber structure. While it is a slight digression, it might be suggested that if every fire test of a proposed type of fire-resisting construction had been carried far enough to determine correctly the salvage after the fire, a good many erroneous conclusions on the subject of fire-proof buildings would have been avoided.

Mr. Turner seems not to have comprehended the real intent of the paper at all; he has set up a straw man and knocked him over, but it is not clear that this has any very direct bearing on the questions discussed. However, an attempt will be made to answer Mr. Turner's objections.

There is decided room for differences of opinion as to whether the type of construction shown by Mr. Turner in Figs. 8 and 9 will be more economical than an equivalent one, using ribs or beams, and a thinner slab. Ingenuity in designing the centering for the ribs might upset Mr. Turner's estimates of cost, without the slightest trouble.

The writer is quite well aware, however, of the extra cost of centering for ribs, and, for that reason, has often preferred and used a plain slab rather than a ribbed construction, even though the slab was quite heavy. But he still maintains that for extensive floors, and heavy loads, there is economy in a ribbed construction; moreover, the conditions of the problem do fix the spacing of ribs within rather narrow limits, as a rule. However, it was not the purpose to treat this question by the laws of maxima and minima, for the

Captain Sewell. reason that the writer has not been able to find any principles governing it in such a way that the skill of a good foreman or designer, expended on the centering, might not be the vital factor in deciding the question, after all. But, assuming the spacing of beams and girders to be fixed, there are such questions as the most economical design of slab and the most economical design of beam, and it was these questions that the writer undertook to discuss—that is, the economical sections of slabs and beams, the bending moments in the two cases being known. In the structure, as a whole, there may be broader questions of economy than in its individual members; but there is an economical design for each of them, and it should not be neglected, for it has a very vital bearing on the whole of the broader question.

Mr. Turner seems also to have missed the essence of the discussion of minimum cost for individual members. If concrete is very cheap, and steel very dear, it may easily happen that the cheapest beam, to carry a given load, may be one in which the concrete is not stressed to anything like its safe limit. On the other hand, it might be possible for concrete to be so expensive, and steel so cheap, that the least expensive beam would be one in which the steel was stressed very lightly, and the concrete to the full limit. Under such conditions, the paper might have to be revised; but, if such conditions existed, it would probably be cheaper to use structural steel; therefore no attempt was made to discuss the question under such assumptions. It is somewhat surprising, however, that this point has not been raised in the discussion of the paper.

As far as the utility of the writer's economic theory is concerned, he has found it useful in his own work, and detailed estimates based on actual designs have proven it correct within its own limits. To make its utility perfectly clear, one might suppose a piece of construction work in which the excavation for footings, etc., yields an ideal material for concrete; there may be nearby a cement plant, from which cement can be obtained at mill prices. Probably the concrete, apart from centering, in such cases, would cost not more than $2.25 per cu. yd. Suppose the cost of steel delivered at the site is very high, say, 5 cents per lb. It does not require a mathematical discussion to show that under such circumstances, economy demands deep beams and slabs, and light reinforcement; but there is an economical limit, and the writer's theory enables the engineer to determine it at once. It would seem that this is worth while, at any rate.

The writer is unable to see the peculiar value of his economical theory to any commercial interest, but as he has never been engaged in any commercial enterprise, there may be possibilities in it which his lack of experience prevents him from seeing.

Mr. Turner lays great stress upon the tests of certain floor slabs, presumably of his own design. Their behavior cannot be explained by any rational formula based on flexure, for the simple reason that they were never subjected to such a bending moment as Mr. Turner figures as due to the load. The secret of their great carrying power is the absolutely unyielding abutments afforded on all sides by the remainder of the floor construction. The writer has had exactly parallel cases in his own work, and has seen them in the work of others. If Mr. Turner had selected an outer panel, next to the wall, for his test, he might have obtained a very different result; or, if he had cut his test panel loose from the neighboring panels—or, if he had tested an entire floor to the extent illustrated in his photographs—it is quite certain that the results would not have been inexplicable, even by the theory of flexure. As a matter of fact, in the panel which was about 16 ft. square, the shearing stress around the outer edges appears to have been not more than 60 lb. per sq. in.—not a dangerous value for really good concrete. The panel, acting as a flat dome, was able to carry the load with stresses probably not exceeding 2 000 lb. per sq. in. The reinforcement below, and the load above, prevented local deformation or buckling. It is very doubtful, however, whether two or three times the load of 900 lb. per sq. ft. would have been required to produce collapse; in fact, the load, as it was, was rather dangerously close to the limit, and, also, under such conditions, collapse is almost sure to be sudden and without appreciable deflection or warning of any kind. Mr. Turner would have added much to the knowledge on the subject if he had carried the test to destruction. Captain Sewell.

The writer's opinions on the subject are derived from the study of one or two actual failures, where conditions were not entirely unlike those shown in Mr. Turner's photograph; but he will be very glad to revise them, if Mr. Turner, by a test or tests carried to destruction, can prove them wrong.

The writer's explanation of the carrying power of the slab is not materially different from that indicated by Mr. Turner himself, in Fig. 11. Both require unyielding abutments; the slightest movement in these would develop the bending stresses at once, and even Mr. Turner will hardly claim that his slabs would have resisted, as beams, the bending moments due to his loads. The writer has used slabs reinforced both ways by plain round rods, and supported all around by beams, as in Mr. Turner's tests; and, where he felt sure of unyielding abutments, he has reduced the bending moments for which the slab was designed, to an extent that he would not recommend for ordinary practice. As Mr. Turner does not disclose his own methods, he may feel the same way about them; but, any one familar with the carrying power of a flat Guastavino dome need feel no surprise at the carrying power of

Captain Sewell.

Mr. Turner's slabs, whatever may be the true explanation. It might also be suggested that steel beams tightly framed in between unyielding abutments, would carry loads as much in excess of their capacities, as based on the theory of flexure, as did Mr. Turner's floor slabs. The writer has often thought that, in insisting upon statically determinate conditions, American designers of steel structures have sacrificed a great deal of reserve strength, especially against local overloads, as well as a very desirable rigidity and a perfectly justifiable economy.

To the writer's mind, however, Mr. Turner's slabs were in a dangerous state of unstable equilibrium under the test loads; a very little fire, or a sudden shock, applied in addition to the loads, would probably have caused collapse of a very sudden and destructive nature. While the writer knows but little of the floor in the building of the Farwell, Ozmun and Kirk Company, to which Mr. Turner refers, he thinks that, in the absence of tests to destruction in both cases, final conclusions and invidious comparisons are not justified.

It is noted that Mr. Turner does not state the working loads for which the floors described by him were designed, nor does he reveal the methods used—all of which would have been much appreciated.

The basis of the writer's belief in attached web members is the very satisfactory practical results shown in their use. Professor Talbot's tests, referred to by Mr. Turner, were hardly fair, because the web members were not long enough, and they were not spaced according to the variation in stresses. The writer used the Warren truss as the analogy, because tests show that the web stresses in reinforced concrete, or any other solid, beam, are inclined at an angle of about 45°, and, as long as the beam is solid, they cannot be made to take any other direction; vertical web members can take up the vertical components of the tensile web stresses, but they cannot change the direction of the resultant stress. As for Mr. Turner's suggestions of an inverted bowstring or a Bollman truss, he would find some difficulty in anchoring his tensile members at the ends. In his application of the writer's economic theory to a plate girder made of two kinds of steel, he deduces results which would be correct if, in steel, it were cheaper to let the unit web stresses vary; but, as a matter of fact, the conditions are entirely different, and the unit stress in the web would be the constant—not the thickness of the web. This application to an impossible—or at least highly improbable—plate girder, is merely another straw man demolished.

The suggestion that the lower part of a beam might be made of clinker concrete was not made with a view of reducing the first cost, but of improving the fire-resisting qualities.

Mr. Wason has made an extensive comparison of different formulas, most of which are convertible, one into the other, by changes in constants, and in the form of the stress-strain curve. The writer has no doubt that the percentages of steel worked out by himself are well within safe limits; he intended that they should be. If economy demanded it, he would not hesitate to increase them by at least 10 per cent. This, in the particular case assumed, would probably have given values more satisfactory to Mr. Wason. Captain Sewell.

However, Mr. Wason's mathematical work is not quite consistent. In his use of Professor Talbot's formulas, he assumes the value of A, overlooking the fact that there is a certain value of A which necessarily follows from his previous assumptions, and which Professor Talbot's formulas afford the means of determining. If Mr. Wason will take the writer's Equations 1 to 5, revise them to suit the assumption of a parabolic stress-strain curve with its vertex on the extreme fiber in compression, and assume $\frac{E_c}{E_s} = \frac{1}{20}$, he will get a set of formulas which are absolutely convertible into Professor Talbot's when developed under the assumptions made by Mr. Wason. He will find, also, that, with such a stress-strain curve, a value of $\frac{1}{10}$ for n in Professor Talbot's formulas is the same assumption as a value of $\frac{1}{20}$ for $\frac{E_c}{E_s}$ in the writer's formulas, revised for the parabolic curve.

Mr. Wason's own formula is based upon impossible assumptions. If the neutral axis is at the middle, the upper half—not the upper third—is in compression. Solving for working stresses, the right line is undoubtedly the correct form for the stress-strain curve. Under these circumstances, the maximum fiber stress in beams designed by Mr. Wason's formula is 667, instead of 500, lb. per sq. in. Moreover, in Mr. Wason's method, there is nothing to indicate that if he had used 100 000 instead of 50 000 for f, he would have obtained a different value of A; in which case, his formula would make the resisting moment of his beam 1 500 000, instead of 750 000 in-lb. Mr. Wason would not consider the beam good for that, but if his formula were written into a building law, some one else might. As a matter of fact, in his formulas, Mr. Wason does not take account of the elastic properties of the materials at all.

At the bottom of page 256,* Mr. Wason makes an interesting calculation, which is a good argument for using a curved form of stress-strain curve, and solving for stresses near the ultimate, since the right-line-working-stress method puts the concrete at a manifestly unfair disadvantage. In Mr. Wason's computation of the

* *Proceedings*, Am. Soc. C. E., for March, 1906.

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compressive force of concrete, under the parabolic assumption, however, the coefficient, $\frac{5}{8}$, should be $\frac{2}{3}$. The coefficient, $\frac{5}{8}$, belongs, however, to a curve much closer to the truth than a parabola, and the lever arm of 0.8125 d is quite safe. There is no doubt that the beam assumed by Mr. Wason would easily carry 2 sq. in. of reinforcement, and that it would not then collapse under less than 1 000 000 in-lb., if properly designed and built.

It seems to the writer that compression in the steel, due to shrinkage of the concrete, is merely the tensile strength of the concrete in another form. It is an additional argument for attached web members.

The Watertown tests indicate that, with increasing age, the ultimate strength of the concrete increases more rapidly than the modulus of elasticity. This increases somewhat the factor of safety, but does not in any way affect the economy of the design.

The tests with loose diagonal stirrups, cited by Mr. Wason, seem to prove very conclusively the existence of the diagonal web stresses, and the necessity for rigid attachment of the web members, resisting them, to the main bars. Mr. Wason is entirely mistaken as to the difficulty of assembling and handling such reinforcement as was used at the War College. The web members were so rigid that the entire reinforcement was easily handled as a whole; the cost, in place, was about 2.4 cents per lb. The concrete was made of small gravel, and poured in, very wet. The writer does not believe that any one of the systems of web reinforcement described by Mr. Wason is as cheap or as easily handled and embedded as that used at the War College.

The equation deduced by the writer for the cost of the variable elements of a reinforced concrete beam is a hyperbola, which, when plotted, would almost coincide with the curve of total cost in Fig. 24, of Mr. Goodrich's discussion.

In answer to Mr. Goodrich's comments, on page 277,* relative to the quantities, s, b, and c, this really raises the question of maximum economy in the spacing of ribs. With a given system of centering, this problem is capable of solution, but it would have to be solved separately for each case. Practical considerations will generally restrict the number of bays into which a given space can be divided, but a mathematical discussion of minimum cost might be useful in each case, as indicating which of the available numbers of bays is most economical.

It seems to the writer that Mr. Goodrich's brick beams had a type of connection between web members and main bars which, when deflection had taken up the slack, became quite rigid, because of

* *Proceedings*, Am. Soc. C. E., for March, 1906.

the friction. If he had set the U's the other way, the experiment would have been more conclusive. As it was, it appears to be quite as much in favor of his contentions as against them. If the stirrups of the beams shown in Plate XXVIII were wrapped closely around the main bars, the same is true of them. Captain Sewell.

Mr. Goodrich, as well as Mr. Turner, expresses some doubt as to the correctness of the ordinary assumptions made in designing reinforced concrete structures. It will not be denied that, for isolated beams, the theory of flexure explains the results of tests fairly well. Slabs reinforced in two directions and supported on all sides may give results a little higher than would be indicated by the theory of flexure as ordinarily applied, and this may be due, in some measure, to the counterbalancing effects of compressive stresses acting at right angles to each other, on the same material. But it is not clear how the two sets of reinforcing bars could relieve each other of tensile deformation in the same way, so that an isolated slab built and supported as described, would probably not carry much more than the breaking loads found by the theory of flexure, rationally applied. Any increase in strength could be allowed for by a judicious reduction of applied moments in designing.

When a slab or beam is rigidly built in as part of an extended structure, however, there is a very great increase of strength under tests applied to only one or two units of the floor system at a time, especially when they are interior units. Exactly the same results have been attainable with steel structures, any time during the last twenty years, but no one has thought it advisable to do so, and count upon them. If not advisable for a tough and ductile material like steel, it is much less advisable for a brittle material like concrete. If dome action is to be counted upon, then build domes, self-contained, each within its own tension ring—not slabs and beams, dependent upon unloaded and undamaged neighbors for capacity to carry, as domes, loads that applied to them, as beams, would instantaneously and completely destroy them. Of continuous girder action, however, the writer would take fairly full advantage; and he is convinced that it would have been advisable to do the same thing in steel structures for these many years.

In answer to Mr. Goodrich's claim that a sufficiency of tests is available to settle disputed points, it might be suggested that the manifest differences of opinion among those conversant with the subject is pretty good proof to the contrary. Beams exactly alike in all particulars except as to the type of web reinforcement and its method of attachment to the main bars, should be tested accurately, in a testing machine, with numerous points of contact, well distributed throughout the span, to prove whether the writer's theory

Captain Sewell. of web reinforcement is or is not correct. Practical tests are not sufficiently accurate to be conclusive and to remove the question from the domain of opinion to that of established fact.

The writer desires to express his appreciation of Mr. Goodrich's very discerning, fair, and frank discussion.

Mr. Thacher's objections to the writer's proposed multiplied loads would apply to bridges and other structures in which the dead load is the principal load. A fair average case in a building would be one in which the dead and live loads are about equal. The writer's proposition, in such a case, involves a safety factor of 2½, based on conditions at a stage materially short of collapse. It is very doubtful whether the floor systems of most steel-frame buildings have any such factor; the tile arches in common use would collapse before the stress in the steel beams reached the elastic limit—and this would fix the factor at very little more than 2.

If reinforced concrete is made a little safer than the type of structures it is trying to displace, it is sufficient. Where the dead load becomes very great, however, there should be adopted a sort of sliding scale, whereby the factor of safety, based on total loads, would never be less than 2½. For structures subject to shock and vibration, it should be increased.

The writer naturally dissents from Mr. Thacher's views as to the adequacy of the concrete for binding main and web members together. However, if there is any economy in omitting the surplus concrete, in a completely reinforced beam, there is no objection to doing so, provided adequate fire resistance is not sacrificed.

In reply to Mr. Forchhammer, attention is called to the fact that the stress-strain curve used in deducing Equations 1 to 5 is correct only on the assumption of a certain maximum stress in the concrete; to allow this stress to vary as Mr. Forchhammer does, and still use the equations with the constants deduced by the writer, is manifestly incorrect. Mr. Forchhammer, in arriving at a beam of minimum cost, goes through all the tentative calculations, which the writer tried to avoid, and deduces no rule of general application.

The writer cannot see that his use of the words, "maximum allowable percentage," is misleading when they are read with the context, wherein it is plainly stated that all percentages are determined so as to cause the beam to fail by failure of the steel.

"Theoretical economy based on relative costs," is attained when the cost of the steel and the cost of the concrete above it are equal. If this gives so great a percentage of steel that the concrete will fail before the stress in the steel reaches the elastic limit, the beam is not able to resist the moment used in determining the area of the steel. A deeper or wider beam of greater cost is demanded. There-

fore, the beam of least cost, determined solely by consideration of the relative cost of the two materials is not adequate, from a structural point of view, and cannot be used. The maximum attainable economy, of course, results from using the "maximum allowable percentage" of steel. When more than this is used, the strength of the beam is increased, but not in proportion to the increased area of steel nor to the increased cost. It may not be quite accurate to say that the increase in the steel is all "wasted," but it is certainly not economically used. There may be cases where dead weight must be avoided at any cost—or where minimum thicknesses must prevail—in which the design must be based on the use of very large percentages of steel; but ordinarily this is not necessary. Where the strength of the concrete determines the failure of the beam or slab, the failure is apt to be sudden, and to come without much warning. This is a very undesirable kind of failure, and should be avoided, if possible. As already pointed out, if concrete is so expensive that it is cheaper to design on the basis of the strength of the concrete, it is probably still cheaper to use structural steel and some form of fire-proof floor arch, such as hollow tiles. Captain Sewell.

In reply to Mr. French, there is no objection to designing by the straight-line formulas, for working stresses, provided values are assumed for $\frac{E_c}{E_s}$ and F, which will be as near the truth as possible, and at the same time, will permit of the use of reinforced concrete with a safety factor only a little greater than that existing in structural steelwork designed to carry the same loads. Building departments will not ordinarily approve designs frankly made with such values of $\frac{E_c}{E_s}$ and F—especially the latter—yet they will pass lighter designs made in accordance with a purely empirical formula in which a fictitious working stress of 500 lb. or less is one of the factors, and an utterly impossible value for the area of compression is another. The writer frankly admits that his chief reason for recommending the method of multiplied loads and ultimate stresses is the hope that it may the more easily lead to designing by honest formulas, with reasonable factors of safety. The writer often designs for working stresses, but, fortunately, does not have to reckon with any building department.

The writer is much indebted to Professor Church for working out the more general solution of his problem in maxima and minima, thereby avoiding the slight inaccuracy of assuming a fixed value for the coefficient, h. That the writer himself evaded this issue should, at least, relieve him of the suspicion of being a mathematical gymnast, which seems to have been aroused in the minds of some by the very modest mathematical exercises contained

Captain Sewell. in the paper. It should be pointed out, however, that the more rigid method followed by Professor Church, when applied to the practical case in which there is a minimum, states the conditions of minimum cost in terms of relative stresses in the steel and concrete. To determine the actual area of steel in the design of minimum cost would then involve the solution of Equations 1 to 5. The writer's method does not give, with rigid accuracy, the theoretical minimum; but it comes close enough for all practical purposes, and gives the result in more convenient form for use.

The question of combined thrust and moment raised by Mr. Leffler is one of extreme interest. This was not discussed in the paper because the writer does not believe in relying upon arch action in floor systems as ordinarily designed. Therefore, he confined himself to a consideration of stresses due to bending only. The simplified formula recommended by the writer is just as applicable to working stresses as to ultimate stresses, but, as it is concerned with the steel alone, it would not serve for the combined thrust and moment problem, without the use of special values of $\frac{a}{d}$.

Replying to Mr. Hill, the writer would not recommend the duplication of his own mathematical work in any case; the general rule that minimum cost will result when the cost of the steel is as nearly as possible equal to that of the concrete above it, is the only practical result of the application of the calculus, and it is merely one point that should be kept in mind, along with the cost of centering, variations in the prices of steel and cement, caprices of the labor unions, etc.

In reference to the dehydration of cement, mentioned by Mr. Hill, if attached web members are not used, concrete which is at all dehydrated would certainly not be reliable for transmitting stresses into the steel. Even if attached web members are used, the concrete surrounding the main bars will take some stress; although this action is not counted upon, it cannot be entirely avoided. Dehydrated concrete would be very apt to crack and fall off, under such circumstances; in any case, unless it is removed and good material substituted, the damage is not wholly repaired, and the structure is not as good a fire risk as it was before.

The last paragraph of Mr. Hill's discussion is of the utmost importance, and should receive most careful consideration.

Mr. Shearwood also raises some extremely important points, which should be settled by careful experimental investigation. The point about the effect of repeated loads is one of great interest; personally, the writer believes that experiments along this line will demonstrate the value of attached web members, but frankly admits that, thus far, this is only an opinion.